JOURNAL

of the

AMERICAN CONCRETE INSTITUTE

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with the direction of the PUBLICATIONS COMMITTEE: W. A. SLATER, chairman; DUFF A. ABRAMS, P. H. BATES, S. C. HOLLISTER, A. R. LORD, A. N. TALBOT, BENJAMIN WILK, HARVEY WHIPPLE, Secretary.

Papers: Original papers, furthering the objects of the society, are invited. They should be addressed to the Secretary, and their consideration by the Publications Committee will be facilitated if: (1) a brief but comprehensive outline or summary precedes or at least accompanies the completed paper; (2) manuscript and photographs are available in duplicate or triplicate; (3) original tracings or other drawings, from which copies are readily made, are available.

Effort will be made to advise authors promptly of decisions as to availability for publication. Acceptance of a paper does not imply the acquiescence of critics (whose advice is sought prior to acceptance) or of the Institute in opinions which it expresses nor the acceptance of data or practice which it records. Institute authority attaches only to standards adopted as provided in the By-Laws. The publication of a committee report implies merely the cooperative effort of an appointed group within the Institute. Acceptance of a paper or report for publication does not commit the Program committee in reference to the annual convention.

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American Concrete Institute

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JOURNAL
of the
AMERICAN CONCRETE INSTITUTE

News Letter

THE CONVENTION PROGRAM

SIX MAIN topics, in five technical sessions, will occupy the Institute at its 26th annual convention, Roosevelt Hotel, New Orleans, Tuesday, Wednesday and Thursday, February 11, 12 and 13.

In establishing the principle of water cement ratio with its far-reaching developments in job control, the industry has run a swift circle of thinking and doing and pauses now at allied considerations.

The mix does not tell the whole story —it is what happens to the mix that has a very considerable influence on the final integrity of the resulting concrete.

The manipulation of the mix and the conditions of curing have large consequences.

These considerations provide two of the main topics.

H. F. Gonnerman is completing what his organization tells us is a very important paper on curing conditions, scheduled for publication in the JOURNAL before the convention and for formal presentation and general discussion at the convention. Mr. Woodworth, also of the Portland Cement Association, has in preparation a paper on a special phase of curing—reviving with new data the one-time lively interest in high pressure steam curing for concrete building units. Good practice in curing concrete where nature is all against you, will be the subject of a report from Committee 604, Winter Concreting Methods, R. C. Johnson, Author-Chairman. Critic members of this committee include men who have had real experience in fighting the mercury's downward trend on important jobs—W. F. McNaughton, L. J. Street, I. E. Burks, R. B. Young, Nelson Doe. Mr. Burks wrote the other day in caustic reply to our urging that he hurry up a paper he is preparing: "The first rule for winter concreting is to stay out on the job and nurse the concrete, not inside writing papers." But must one be done to the complete exclusion of the other? If so, how is the Institute to take its preparatory course in that kind of "nursing?"

"Portland cement meeting the standard specifications" is a word formula that has encouraged the view that all cements which meet standard specifications can be depended upon to behave similarly. Gradually the industry has come to know that these two things are not necessary corollaries. The report of Committee 202, Variations in Standard Portland Cements, by P. H. Bates, Author-chairman, in the November JOURNAL lays the foundation for interesting convention discussion.

A session on specifications—a new kind of specification—will consider the report of Committee 502 (November JOURNAL) "Concrete Work on Ordinary Buildings" and probably also another report still in the hands of Committee 506 "Concrete Work on the Small Job" ("where inspection is occasional only and the contractor presumed to be honest"). Both specifications were initiated by the same author-chairman, Arthur R. Lord. Allied to these, particularly to No. 502, is a specification for "Fabricating and Setting Reinforced Steel" (Committee 503, William F. Zabriskie, Author-chairman). Mr. Zabriskie includes as addenda a steel setters primer, which aims to instill first principles as to the how and why of steel in concrete. This specification is now up to the critic members of the committee. Specifying and making centrally mixed concrete are vital considerations these days. Miles N. Clair, Author-chairman of Committee 504, Specifications for Centrally Mixed Concrete, and Frank I. Ginsberg, Chairman Committee 602, Design and Operation of Central Mixing Plants promise interesting contributions.

A session on building design and construction will include a statement of the work ahead of Committee 105, Reinforced Concrete Column Investigation, W. A. Slater, Chairman; the report of Committee 405, Economics of Tall Building Design, R. R. Zipprodt, Author-chairman; the report of Committee 307, Deflection of Reinforced Concrete Members, T. D. Mylrea, Author-chairman and probably another contribution of outstanding possibilities in the building construction field.

The manufacturers of concrete products are to consider two chief topics: the use of color, based on a report of Committee 703, Raymond Wilson, Author-chairman and the report of Committee 707, Benjamin Wilk, Author-chairman, on single or multiple shift operation—a theme likely to develop general discussion of plant design and operating problems.

Of more than usual interest are the prospects of a session in which flood prevention and shore protection tie the interests of the Institute into the region of the 26th annual meeting. Capt. Helmer Swenholt, Corps of Engineers, U. S. A., will discuss the design and construction of the Bonnet Carre Spillway (to divert flood water around New Orleans); Lieut. Morris W. Gilland, Corps of Engineers, U. S. A., will describe the making and placing of concrete revetment mat; J. D. Converse, consulting engineer, Mobile,Ala., the use of concrete in important shore protection projects. Also of interest in the southland will be a paper by J. E. Hough on cementing oil wells.

The program for the annual dinner will be different—with a New Orleans flavor.

ABSTRACTS

THE PUBLICATION of abstracts of current periodical and bulletin literature significant in the field of cement and concrete will begin in the JOURNAL for January, 1930.

Abstracts will constitute a separately printed section, with a series of page numbers independent of both *News Letter* and *Proceedings*, but bound with these following the papers and reports —next to back cover.

Abstracts will be in 8 pt. type in two narrow columns to the page as in *News-Letter* make-up.

It is expected that for some months the work will fall short of that 100% coverage of the literature which is the aim of the effort.

There are two major objects:

(1) To supply to members a timely digest of the current information of the field;

(2) To build up a reference source to which students of concrete may go with a feeling of confidence that nothing significant, published in the period covered by the work, is escaping them.

The first object is in *some* degree attained at the outset. Whatever is published—if covering only so little as 50 per cent of the current literature available, will be just that much more than no abstracts at all. There is at present no generally available service specializing in the field of cement and concrete. As this first object is attained in increasing extent, as is the Institute's very serious and now constant purpose, the second object of thorough dependability will be the natural outcome. As a seal of good faith on the character of this effort announcement will be made within the next few months of a standing offer open to all Institute members (except the regular abstractors who might be responsible) to claim a triple space rate reward for supplying an abstract of any significant contribution—any periodical literature, which has escaped the Institute's *Abstract* pages for three issues after its original publication. It is a matter of daily concern in the Secretary's office so to perfect the abstracting organization as to justify making an offer of that rather confident character, sometime early in 1930.

PERSONNEL

To accomplish this task, the Institute has enlisted some of its ablest talent. First there is the new committee on Abstracts (responsible to the Publications Committee): J. C. Witt, Chairman, W. D. M. Allan, Theodore Crane, H. F. Gonnerman, Rexford Newcomb, F. E. Richart.

Dr. Witt, a chemical engineer, well known in the field of cement technology, knows not only the field of the Institute's subject matter but is conversant with the best abstracting practice. As one of the assistant editors of *Chemical Abstracts*, an institution of vast detail and thoroughness, built up through many years of effort under the editorship of E. J. Crane, Dr. Witt is in charge of the section devoted to "Cement and Other Building Materials." The Institute was fortunate in finding him willing not only to take the chairmanship of its new Committee on Abstracts and to become responsible for the work done in the section on Materials (cement, aggregates, steel and other constituents of and accessories to concrete) but in a general way to give generous advice and assistance in

the preliminary work of organization of the entire abstracting effort.

No one is better fitted than H. F. Gonnerman, Manager of the Research Laboratory of the Portland Cement Association, for general supervision of abstracting in the division on Properties of Concrete, which will be his special responsibility as member of the committee.

Rexford Newcomb, Professor of the History of Architecture, University of Illinois, who, some months ago became co-chairman of the Institute's committee department 400, Architectural Design, will make it his aim as member of the Committee on Abstracts, to give to Institute members everything currently available on architectural design in relation to concrete. Not only will he be responsible for abstracts in this field, but with some assistance available at the University and elsewhere is actually organizing the abstracting, sifting the contents of publications in several languages for things of interest and importance to Institute membership.

F. E. Richart, Research Associate Professor of Theoretical and Applied Mechanics, University of Illinois, very confident of the importance and great usefulness of the Institute abstracts, has taken charge at some sacrifice, of the division of Engineering Design, several important publications in this field being abstracted under his supervision.

Theodore Crane, Associate Professor of Building Construction at Yale University, has very appropriately taken hold of work on Field Construction, the practical workings of concreting on the job. Professor Crane, a thorough student, an analyst and writer in the construction field, is especially fitted to judge the adequacy of work done by abstractors in this field for the Institute.

To contribute to the work of the Committee on Abstracts a thoroughgoing sponsorship for the field of Shop Manufacture there seemed no more likely candidate than W. D. M. Allan, manager of the Cement Products Bureau of the Portland Cement Association, already serving the Institute as co-chairman of committee department 700, Shop Manufacture. Mr. Allan, with his bureau staff, is probably in closer, more timely touch with every noteworthy development in the "products" field than any one else in or out of the Institute. He will review the abstracts in that field and recommend whatever steps may seem necessary to give to the concrete products manufacturers in the Institute (now representing nearly as large a percentage of Institute membership as the contractors) a thorough coverage of the literature bearing on their work.

THE TASK

This seems no place to list all those Institute members who have consented to take on some measure of responsibility in the actual abstracting. Their names will be signed to the abstracts which they prepare and members generally will be made acquainted with their work. To find the right men for the work has been a rather elaborate problem in personnel. It is not enough to discover interested readers in certain fields of interest; it is not enough to find intelligent readers. It is desriable to find men to abstract literature for others who have that rare knack of reducing a long story to a short one, of distilling the essence from 5000 words and putting that essence into 100 words without distortion. The task is the more rigorous since the original intent of the author is the thing

to be preserved without dilution in the opinion of the abstractor. The rule laid down is that the abstract shall effect a maximum abridgment of the original document while preserving its essential quality. This requires good judgment of values and a fine economy of expression.

Some of the abstracts already filtering in to the Secretary's office, digesting papers and reports first published in November (The Institute work begins with November literature) indicate not only a good grasp of the scheme but of the subject matter. Here and there an abstractor condenses too much—sometimes not enough. The earliest returns indicate that the Institute is fortunate in being able on short notice to find in its own ranks such a capable corps of abstractors.

Abstractors are to be paid for their work at space rates as published. While the proposed rate is more than is usually paid for such work, it is far from enough to weigh heavily, if at all, with the men who are doing the work. Most of them are doing it as a bi-product of their own interested reading (which becomes a regular obligation) and as a means of aiding their society in an important new service.

Assignments in many instances are now too broad, individuals being called upon to cover temporarily the whole contents of periodicals so far as they relate to cement and concrete. This may be anything from the constitution of portland cement to job procedure in the construction of a dam. At the same time the abstractors whose help is already assured include many specialists of high authority in their own right.

As the organization of the work proceeds the abstracting will tend more and more to the specialists. Assignment of whole periodicals to individuals (except highly specialized periodicals) will largely give way to a policy of breaking up the periodicals and the assignment of papers and reports to those fitted to give Institute members the best digests. The publications in the various language groups will yield more slowly to this arrangement since the usual "garden variety" of translation is unsuited to highly technical subject matter. Assignments must be made with a view not only to the abstractors familiarity with the technical language of the foreign tongue to be reduced to its English technical equivalent in highly abridged form, but with a view also to his easy conversance with the subject matter of the various divisions into which Institute abstracts, for convenience, will be grouped.

The work thus involved in a thorough-going effort to digest all periodical literature that has a bearing on the Institute's field, is set down here for all Institute members for three reasons: first, as a matter of general interest to those who are to benefit by it; second, and most important, as a means of finding recruits for the corps of abstractors (what branches of the literature do they read regularly? to what periodicals do they have regular access? in what technical languages do they feel at home?—let them forget a natural modesty and reveal themselves) who will be available for special assignments in the future; third, to allay a possible mild impatience in a few who may feel that to publish a monthly abstract service in a highly prolific and highly diversified field, it is merely necessary to begin writing abstracts. About 150 periodicals and regular bulletin sources in half a dozen languages have so far occupied the attention of the Committee on Abstracts. The

personnel so far recruited, is inadequate to the undertaking without continuing to overload some individuals—the personnel is however of exceptionally high calibre. More is needed.

INTRODUCING MR. JENNINGS

To systematize and edit Institute abstracts, to set up and follow-up, in all its meticulous detail, the actual labor of abstracting—collecting, sorting, indexing and publishing, the Institute will augment the staff of the Secretary's office beginning January 1. Dwight F. Jennings, an engineer whose chief concern has been with developments in the field of concrete for ten years—1920-1925 associate editor of the magazine "*Concrete*" and more recently with The Huron Portland Cement Co., Service Bureau, Detroit—will assist the Secretary in the editorial conduct of the JOURNAL and will be in immediate charge of *Abstracts*.

* * *

NEW ORLEANS

MEMBERSHIP of the Institute is not now large in New Orleans but its citizens are proud of their city—its traditions, its legends, its characteristics which set it apart from other cities; they are proud of those features which visitors find picturesque—and most of all they are hospitable. Rumor, from other conventions, has it that it is hard to get any work done in the meetings—there are so many competing interests.

As an outgrowth of a preliminary meeting in May when the Secretary was in New Orleans for preliminary convention arrangements, President Boyer has appointed a New Orleans Convention Committee under the chairmanship of J. L. Porter, of the New Orleans Sewerage and Water Board; with Walter B. Moses, treasurer of the New Orleans Engineering Society as Vice-Chairman and W. L. Macatee, District Engineer Portland Cement Association, Secretary.

The full membership of the committee is as follows:

Scott Thompson, Vice-Chairman
W. L. Macatee, Secretary
Walter B. Moses
F. H. Fox
F. A. Ruth
George Hammett
Selis Seiferth
R. B. Roessle
Walter Jahncke
W. C. Griffen
John Klorer
Ole K. Olsen
Dr. H. S. Shilstone
John Riess
John F. Coleman
Samuel Young
W. B. Davey
John T. Eastwood
Bryson Vallas
Harry Jacobs
Harry B. Hinderlite
Prof. W. B. Gregory
Prof. Donald Derickson
Samuel Fowlkes, Jr.
Major W. H. Holcomb
Major R. F. Fowler
J. L. Porter, Chairman

This committee will see to it that matters of local arrangement for the Institute meeting are really arranged so that visitors the week of February 10 will make the most of the occasion.

* * *

NEW DIRECTORY

A NEW Directory is in preparation for 1930. Corrections of entry and changes of address will be in time for consideration if received by the Secretary not later than January 15.

COMMITTEE TASKS—PAST, PRESENT, FUTURE

TO INCREASE its output of technical information beyond the capacity of our annual convention the Institute adopted the JOURNAL plan of publication.

To meet the need for a better organized and more intensely worked technical program and at the same time meet the "copy" necessities of the monthly publication, when once it should be established, the Board of Direction last February adopted the report of the Advisory Committee, A. R. Lord, chairman, which set up a new committee organization to replace the old one. Effective units of the old organization were retained in the new with a few changes in personnel, but under a plan essentially new in certain elements:

(1) With a few exceptions committees are assigned not to fields of interest but to tasks—most of them to immediate tasks. They were assigned subjects for review; asked to prepare summaries of available information with conclusions indicative of preferred practice; to prepare statements on moot subjects, to recrystalize thought on them, or as a basis of discussion through which some clarifying effect might result; or to prepare specifications. At bottom the assignments, once "covered," would constitute an inventory of what we know and a point of departure in going after what we do not know.

(2) The scheme of new committee organization provided in many of the new working units, an increased incentive. It pointedly put the major responsibility on an individual. He, the author-chairman, not the committee, was asked to initiate a report. Professional pride of performance—of authorship—was given some rein. In general (there are exceptions) a man puts better work into a paper he is called upon to write than into a committee report on which he is asked to co-operate. In the paper he can put vigor and spontaneity—he can look far toward the horizon. In the usual committee report the best he thinks has its wings clipped in compromise. The new committee procedure seemed destined to call out the author-chairman's best without relieving him of the necessity of defending that best against the views of appointed critics—the members of his committee. Nor is the responsibility of the critics covered by a blanket of joint action. If the critic does not approve he may, in fact he *should* object and register that disapproval along with the original document.

(3) A third important consideration in the new scheme is "tenure of office." Assignments mostly, being definite, restricted and immediate, the committee goes out of business when the job is done. The same personnel or a part of the same personnel may be given a new, perhaps a related job. Workers go on working as will always be their reward, but committees die. Appointments are on a 12-month basis.

(4) Then there was a very definite thought to provide an enlarged "working population" in the Institute. Certain dependable wheel horses are worked and worked over and over again in almost all organizations. It gets to be a habit to have a certain few do all the jobs. It is a bad habit when it leaves many other good men without jobs. The real life of any organization bears a very definite relation to the extent of its *working* not its passive membership.

(5) Behind the entire committee scheme was a thought to make a start toward the accumulation of the basic raw material of two volumes which should sometime bear the seal of the Institute: A Digest (of available knowledge of concrete); A Manual of Practice.

(6) To give some kind of order to the committee work; to give it, as a whole, eyes to scan the various fields of interest for new tasks, new problems and new personnel, a departmental system was inaugurated; each of nine departments to which committees were assigned, having a chairman who should be a member of the Advisory Committee and co-chairmen who would supplement his view of each part of the field.

Ten months have passed since the inauguration of the new plan. Soon now the Advisory Committee will go to the Board of Direction with a new set of recommendations. At least thirty new committees with new definite tasks are in the offing. It seems the time has come for members at large to have something to say.

How would they get the Institute's work done?

How would *you* have it done?

What definite task have you to suggest?

What likely additions to the Institute's *productive personnel* are now jobless so far as the committee program is concerned?

These questions are the live issues. In their answer lies the possibility of greater usefulness of members to the Institute and of the Institute to members.

Write the Secretary for the consideration of the Advisory Committee.

COMMITTEE REVIEW

HERE let us review briefly the committee work of the Institute under the new plan organized last winter.

ADVISORY COMMITTEE

Chairman—Arthur R. Lord
Secretary—Harvey Whipple

W. A. Slater (Chairman Publications Committee)
S. C. Hollister (Chairman Program Committee)
F. R. McMillan (Chairman Dept. 100, Research)
R. W. Crum (Chairman Dept. 200, Materials)
A. W. Stephens (Chairman Dept. 300, Engineering Design)
Alfred S. Alschuler (Chairman Dept. 400, Architectural Design)
A. B. Cohen (Chairman Dept. 500, Specifications)
R. B. Young (Chairman Dept. 600, Field Construction)
C. A. Wiepking (Chairman Dept. 700, Shop Manufacture)
R. E. Davis (Chairman Dept. 800, Use Requirements)
F. H. Jackson (Chairman Dept. 900, Joint Efforts)

PROGRAM COMMITTEE

Chairman—S. C. Hollister
Secretary—Harvey Whipple

A. E. Lindau
R. J. Wig
A. R. Lord (Chairman Advisory Committee)
W. A. Slater (Chairman Publications Committee)

PUBLICATIONS COMMITTEE

Chairman—W. A. Slater
Secretary—Harvey Whipple

Duff A. Abrams
P. H. Bates
A. N. Talbot
Benjamin Wilk
A. R. Lord (Chairman Advisory Committee)

S. C. Hollister (Chairman Program Committee)

RESEARCH

Department 100, Research, is composed of F. R. McMillan, Chairman; H. J. Gilkey and F. E. Richart, Co-Chairmen, with the following committees:

101, Survey of Current Research, H. F. Gonnerman, Chairman; F. E. Richart, Secretary. A reorganization of the committee of long standing which has made valuable detailed reports each year, summarizing research work done and in progress as reported by institutions reached by the committee's annual questionnaire. The report for the current year is in progress and probably will be available for publication early in 1930. Members:

J. C. Witt	F. M. McCullough
F. H. Jackson	J. R. Shank
D. V. Terrell	C. E. Proudley
H. J. Gilkey	F. O. Anderegg
H. F. Clemmer	

102, Volume Changes in Concrete. Raymond E. Davis, Author-Chairman; H. J. Gilkey, Charles C. More, Critics. The Author-Chairman has prepared, in line with his assignment, a summary of available knowledge on volume changes in concrete. It is now in the hands of the critics.

103, Effect of Curing Conditions. A. A. Anderson, Author-Chairman. This committee was asked for a summary of available knowledge, and the report is assured early in 1930. Critic members of the committee have not been named.

104, Permeability of Concrete. L. G. Lenhardt, Author - Chairman, with Charles H. Schuler, M. O. Withey, Ira L. Collier and R. H. Schroeder, critic members. The Author-Chairman is engaged in the final arrangement of many data, the result of digesting literature. The report will doubtless be ready for submission to the critic members of the committee before the end of this year.

105, Reinforced Concrete Column Investigation. W. A. Slater, Chairman. Detailed announcement of the work of this committee, making a new series of tests of reinforced concrete columns which will probably occupy the committee for a year or two has previously been made in these columns. Members:

R. L. Bertin	F. E. Richart
R. D. Snodgrass	W. S. Thomson

MATERIALS

Department 200, Materials, consists of R. W. Crum, Chairman and Prof. E. O. Sweetser and R. W. Johnson, Co-Chairmen with the following committees:

201, Aggregate Specifications. H. F. Clemmer, Chairman. The committee has found it necessary to suspend work temporarily on the perfection of aggregate specifications in order to prepare a new program for a fresh line of attack involving investigational work before specifications may be further developed. Members:

F. H. Jackson	N. C. Rockwood
L. E. Williams	Fred Hubbard
Herb F. Kriege	A. T. Goldbeck
R. L. Bertin	Stanton Walker
H. S. Mattimore	H. J. Love
Duff A. Abrams	

202, Variations in Standard Portland Cements. P. H. Bates, Author-Chairman. A report prepared by Mr. Bates was published in the JOURNAL for November together wtih discussion by one of the critic members of the committee, Thaddeus Merriman. Other critic members, T. Thorvaldson, Raymond E. Davis and G. M. Williams have discussions under consideration

ENGINEERING DESIGN

Department 300, Engineering Design, consists of A. W. Stephens, Chairman and Prof. Hale Sutherland and Ralph G. Shankland, Co-Chairmen, with the following committees:

301, Simplified Rigid Frame Design, Hardy Cross, Author-Chairman. Critic members, F. E. Richart, Clyde T. Morris, Albert Smith. Professor Cross' report, approved by the critics, is published in this month's JOURNAL.

302, Design of Two-Way Slabs on Beams and 303, Design of Spiral Type Columns, were authorized by the Board of Direction but organization was not effected.

304, Fire Resistance Standards in Design, N. D. Mitchell, Author-Chairman, with the following critics: D. H. Dixon, H. M. Robinson, D. E. Parson, C. A. Menzel. Work is in progress.

305, Structural Loads and Reductions in Design, R. L. Bertin, Author-Chairman with George Thompson, H. S. Hein, H. M. Howe, as critic members. Mr. Bertin expects to finish his report and have it ready for critic members at a very early date.

306, Permissible Openings in Construction. Albert Smith, Author-Chairman, G. A. Maney, A. E. Lindau, critic members. The report was published in the November JOURNAL.

307, Deflections of Reinforced Concrete Members, T. D. Mylrea, Author-Chairman; critic members not appointed. Professor Mylrea's report is assured early in 1930.

ARCHITECTURAL DESIGN

Department 400, Architectural Design, consists of Alfred L. Alschuler, Chairman, with Rexford Newcomb and O. F. Johnson, Co-Chairmen, with the following committees:

401, Portland Cement Stucco Finishes, W. D. M. Allan, Author-Chairman with the following critic members: Allan Stubbs, C. S. Downs, F. O. Anderegg, Arthur Ware, J. C. Pearson. Report of the committee was published in the November JOURNAL.

402, Concrete as a Sculptural Medium, Lorado Taft, Author-Chairman. Mr. Taft and his associates have a report in preparation.

403, Monolithic Concrete Surfaces, Nathan C. Johnson, Author-Chairman. Critic membership not fully organized; report in preparation.

404, Composite Concrete Structures, Benjamin C. Shapiro, Author-Chairman. Committee not fully organized.

405, Economics of Tall Building Design, R. R. Zipprodt, Author-Chairman. Critic membership not fully organized. Report is assured as available early in 1930.

SPECIFICATIONS

Department 500, Specifications, A. B. Cohen, Chairman, with P. J. Freeman, Co-Chairman, with the following committees:

501, Standard Building Code, A. W. Stephens, Chairman; R. R. Zipprodt, Secretary. This committee's assignment is the perfection of "Tentative Standard Building Regulations for Reinforced Concrete" as presented to the Institute in 1928. A further report is expected in 1930 or 1931. Members:

R. L. Bertin
H. P. Bigler
R. B. Bradbury
Frank E. Brown
M. E. Capouch
Prof. Theo. Crane
M. E. Hibbs
H. N. Howe
A. E. Lindau
A. R. Lord
Prof. T. D. Mylrea
Prof. F. E. Richart
Prof. W. A. Slater
W. S. Thomson
Prof. L. C. Urquhart
Prof. H. M. Westergaard
N. D. Mitchell
Prof. Hale Sutherland

F. R. McMillan Prof. Hardy Cross
R. B. Young

502, Mixing and Placing Concrete on Buildings, Arthur R. Lord, Author-Chairman, with A. S. Douglass, A. B. MacMillan, W. F. Way and J. Thumley as critic members. Report published in the JOURNAL for November.

503, Fabricating and Setting Reinforcing Steel, W. F. Zabriskie, Author-Chairman. Critic membership not fully organized. Mr. Zabriskie has completed draft of specifications asked for and within a few days it will be under consideration by the critic members.

504, Specifications for Centrally Mixed Concrete, Miles N. Clair, Author-Chairman with the following membership: P. J. Freeman, W. E. Hart, R. S. Greenman, C. E. Nichols, Manton E. Hibbs, Alexander Foster, Jr., Frank I. Ginsberg. This committee has specifications in preparation for early availability.

505, Reinforced Concrete Chimneys, E. A. Dockstader, Author-Chairman. Mr. Dockstader is completing the work of an old Institute committee, specifications for the design and construction of concrete chimneys.

506, Concrete Specifications for the Small Job, Arthur R. Lord, Author-Chairman. Critic membership of the committee is being organized for consideration of specifications which Mr. Lord has already prepared.

FIELD CONSTRUCTION

Department 600, Field Construction. R. B. Young, Chairman with R. C. Johnson and Nelson L. Doe, Co-Chairmen and the following committees:

601, Measuring Materials for Concrete, R. B. Young, Author-Chairman. The report of this committee depends upon a large amount of field investigation work and data to be gathered.

602, Design and Operation of Central Mixing Plants, Frank I. Ginsberg, Author-Chairman. Report is in preparation on practical considerations implied by the title of the committee for early presentation to the Institute.

603, Developing Latent Strength of Concrete, H. F. Gonnerman, Author-Chairman. A report on the possibility of putting strength into concrete which has been abused in fabrication or in curing is an interesting subject which Mr. Gonnerman had to put aside temporarily in favor of a more pressing assignment on which he is engaged.

604, Winter Concreting Methods, R. C. Johnson, Author-Chairman with the following critic members: W. S. McNaughton, L. J. Street, I. E. Burks, R. B. Young, Nelson L. Doe. The Author-Chairman is at work upon a report which will be available for the critic members of the committee in the very near future and with a view to presentation at the 1930 convention.

SHOP MANUFACTURE

Department 700, Shop Manufacture, C. A. Wiepking, Chairman, C. F. Buente and W. D. M. Allan, Co-Chairmen, with the following committees:

701, Standards for Concrete Masonry Units, Benjamin Wilk, Chairman, E. J. Lantz, Secretary. This committee will probably have no report for presentation on development of specifications for concrete masonry units. Members:

C. J. Herzog George O. Payne
Einar Christensen A. G. Swanson
E. W. Dienhart D. E. Parsons

702, Prevention of Crazing, P. H. Bates, Author-Chairman with the following critic members, John J. Earley, J. C. Pearson, E. D. Boyer, F. O.

Anderegg. The committee has no report in immediate prospect.

703, Color in Concrete Products, Raymond Wilson, Author-Chairman. Critic members, O. A. Malone, Paul Formigli. Mr. Wilson has a summary of available knowledge in connection with color in concrete products in preparation for early availability.

704, Cast Stone Standards, Louis A. Falco, Chairman, C. G. Walker, Secretary. This committee is charged with the development of specifications adopted tentatively by the Institute last winter. Members:

Duncan Thomson P. H. Bates
George J. Eyrick J. W. Lowell
Herman Fraunfelder

705, Weather Resistance of Concrete Masonry Units, C. A. Wiepking, Author-Chairman, with W. D. M. Allan, Einar Christensen and Dan F. Servey, critics. Report from this committee is largely dependent upon data from fresh investigations not immediately forthcoming.

706, Concrete Pipe, Asa E. Phillips, Chairman. This committee is not fully organized.

707, Plant Design for Single or Multiple Shift Operation, Benjamin Wilk, Author-Chairman. A frankly controversial subject handled in a preliminary presentation by Mr. Wilk will shortly be presented to the critic members.

USE REQUIREMENTS

Department 800, Use Requirements, Raymond E. Davis, Chairman with W. F. Way as Co-Chairman and the following committees:

801, Durability of Concrete, A. E. Lindau, Chairman, F. R. McMillan, Secretary. It is the function of this committee as of its predecessor, the committee on Destructive Agents and Protective Treatments, to essay the thankless task of examination of structures to learn if possible why some stand up and others disintegrate. The committee has made several field surveys in the last year and a report is in preparation. Members:

R. B. Young P. J. Freeman
E. Viens L. W. Walter
G. F. Loughlin

802, Portland Cement Floor Finish, John G. Ahlers, Chairman, W. E. Hart, Secretary. This committee has in preparation for early presentation a report summarizing available information on good practice in the finish of concrete floors for special uses.

803, Disintegration of Concrete, G. M. Williams, Author-Chairman, with T. Thorvaldson, John R. Baylis, D. G. Miller, critic members. The report of the committee prepared by Professor Williams and approved by the critics was published in the November JOURNAL

JOINT EFFORTS

Department 900, Joint Efforts, F. H. Jackson, Chairman, Cloyd M. Chapman, Co-Chairman. The province of this department is such committee work as is undertaken jointly with other organizations or jointly between or among committees of other Institute departments.

901, Concrete Culvert Pipe Standards, B. F. Pease, Chairman for Institute representation. The second progress report of this joint committee was presented to the Institute a year ago.

902, Concrete Roads, Duff A. Abrams, Chairman. Committee not fully organized.

FACULTY MEMBERS

A LETTER FROM ARTHUR R. LORD

YOUR NOVEMBER *News Letter* seems to invite contributions to this department. If the "squib" which follows appears to you to be of interest to other Institute members, you may use it. It is the result of one fragment of a conversation I had very recently with a well known and highly regarded member. We were talking of the "extra-curricula" activities, not of students, but of members of an engineering faculty. I had once joined the entering class—he was well toward the top of such a group. He felt that the spirit at his school was excellent, and steadily improving. Faculty members, with few exceptions, were reaching out vigorously for outside stimuli. They had joined the A. S. C. E., the A. S. T. M. or the A. C. I., and they were contributing to the discussion, attending the meetings, making contacts with the practicing engineer, thinking of his problems—which after all were the problems which their students must be prepared to face. They were definitely keeping out of the well-worn rut of changeless repetition, the deadly monotony of teaching the same thing in precisely the same way, semester after semester. Their courses were annually freshened with new ideas, they wrote new textbooks from their enlarged experience. Many of them engaged, as their time and primary teaching interest permitted, in research both mathematical and physical.

The conversation set me to thinking and figuring. How much of this outreaching was directed to the American Concrete Institute? The Institute would seem to offer an unequalled opportunity. Concrete enters into every engineering field, into the early experience of almost all engineering graduates. Concrete, with its multitude of uses, its many variables leading to great diversity in characteristics controllable for such particular uses—far excelling any other structural material in this respect—surely offers a stimulating challenge to the keen minds of the college instructor, professor and research investigator. The Institute, made up largely of professional men* with nothing to sell save their personal services, would appear to be a natural outlet for faculty expression in this absorbingly interesting field of concrete theory and practice. The assistance of the university man is welcomed by the practicing member of the Institute, as is evidenced by the awards of our Wason and Turner medals and by the membership of our committees. To what extent are our various engineering colleges participating?

The Directory of the Institute offered a possible answer. I soon confirmed A. Edward Newton's earlier discovery that the supply of questions in this world far exceeds the supply of answers. If a member lives in Ithaca, is he necessarily a Cornell instructor?† I assumed that some of the population of Ithaca, even Institute minded, were not of the faculty. In Columbus, it seemed even more certain that there were would-be Institute members not connected with *Ohio State*. Where only the street address was given, therefore, I could not be sure, but with many the University connection was indicated or known to me. In the summary of my search which follows, I have not in-

*See "Who and What Are Our Members" page 17—EDITOR.

†It is unfortunate perhaps that names in the A. C. I. Directory are not all "labeled." If a college professor uses a residence address, we must respect his choice.—EDITOR.

tentionally slighted any institution, but as I do not have the rare pleasure of knowing personally all the Institute members, nor all the faculty members, I am sure to err. Even an approximate summary may prove interesting, as given very briefly below.

I found over sixty colleges represented in the Institute—about five per cent of our total membership is of faculty extraction. Fifteen of these colleges are outside the United States. Some foreign schools are represented by two or more faculty members. I found some surprises. Professor Maney, who I know to be in Evanston, right next door to Chicago, is listed as from North*eastern* University,‡ which is in China—and which is represented in the Institute! There's another Northeastern University in Boston, however, also represented, so Friend Maney does not have to stay in China!

Naturally enough, *Illinois*, famous seat of concrete development and research, leads all other colleges with eleven faculty members in the Institute. Many of them are exceptionally active members. *Purdue* and *Wisconsin*, rivaling *Illinois* in the early development of concrete facts and theories, have fallen greatly, if present Institute membership be a criterion—they have but three representatives, each, although among the six are such leaders as Hatt, Hool, and Withey. Second place is held by *Michigan* with nine, third and fourth by *Cornell* and *M. I. T.* It is surprising how many great engineering schools are represented by two or even one faculty member only. Such colleges could effect a fair exchange with the Institute by a larger representation; their faculty members giving to the Institute their invaluable interest and service and taking back to the campus the inspiration of contacts with others who have joined hands in studying, analyzing, perfecting the applications of this unique and universal engineering material, concrete. If graduates of the colleges were included, as well as teachers, the story might be different, but I suspect that Illinois would still lead. Since a few Institute members are still missing from Who's Who or other readily consulted manuals that give their college affiliations, I have not attempted this larger comparison.—ARTHUR R. LORD.

‡Apologies. We are checking up to see whose *we* looks like *ea*.—EDITOR.

* * *

NEW MEMBERS

THE BOARD OF DIRECTION has approved applications for membership received in the month of November for the following:

Bowman, H. L., Professor of Civil Engineering, Drexel Institute, Philadelphia, Pa.

Erickson, Jr., Arioch Wentworth, Greystone House, Swampscott, Mass.

Hertz, H. P., Routledge & Hertz, Archts. & Engrs., 204-5 Ex. National Bank Bldg., Hutchinson, Kansas.

Licuanan, Jose G., P. O. Box 1589, Manila, Philippine Islands.

Metropolitan Concrete Co., Western Reserve Bldg., Superior & W. 9th St., Cleveland, Ohio. (K. W. McKelvey)

Norred, Jr., C. V., 2115 Jones Ave., Nashville, Tenn.

Providence Transit Concrete Corp., 40 Aliens Ave., Providence, R. I. (R. F. Moyer)

Rule, H. Paul, 909 Walnut St., Columbia, Pa.

THE HONOR ROLL

THE OTHER day came a letter, half way round the world from a region where helpful technical literature on concrete is highly appreciated. It expressed the hope that means might be found to bring a knowledge of the Institute's work to more and more engineers who certainly must be grateful for such opportunities as this society affords. The letter was from a new member sponsored by some other member who had found membership worth while. This correspondent felt isolated from helpful contacts—and was enthusiastic about the Institute's relief of that isolation. All of which served as fresh testimony that isolation in such matters is folly when it can be remedied. Correlated, organized efforts of many get farther much faster than individual efforts in all labors where so many data and so much experience are required to establish a new and better way.

There are mental isolations as acute as those of geography.

The point of this being that the Honor Roll is a means to eliminate isolation.

Those members (to Dec. 1, 1929) who have sponsored three or more new members since Feb. 1, 1929 are as follows:

H. F. Gonnerman	9
W. F. Way	9
I. E. Burks	7
H. B. Emerson	6
Geo. E. Fenkell	6
M. M. Upson	5
E. Y. Bragger	4
F. R. McMillan	4
D. A. Abrams	3
John G. Ahlers	3
W. D. M. Allan	3
L. E. Andrews	3
H. J. Gilkey	3
Hale Sutherland	3
Miguel Villa (Cuba)	3

• • •

WHO AND WHAT ARE OUR MEMBERS

THE INSTITUTE has been trying for three years to effect a complete classification of its members on a self-determination basis.

We sent out (and regularly send to new members) yellow cards for this purpose and to elicit suggestions to learn the trends of membership interest.

So far 52% of our membership is classified on the basis of these cards:

	Per Cent
Engineers	44
Contractors	12
Producers of materials	12
Manufacturers of Concrete Products	10
Architects	7
Building superintendents and inspectors	5
Testing laboratories and engineers	4
Universities, colleges, etc.	3
Producers of machinery and equipment	1
Students	1
Miscellaneous	1
	100

• • •

AMERICAN MINING CONGRESS

THE AMERICAN CONCRETE INSTITUTE'S official representatives at the 32nd annual convention of the American Mining Congress, Washington, D. C., December 4 to 7, were A. T. Goldbeck, Director Bureau of Engineering, National Crushed Stone Assn. and Stanton Walker, Director Engineering and Research Division, National Sand and Gravel Association.

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THE COLUMN TEST FUND

TO DECEMBER 14, $7990.00 is the total of cash received and deposited by the Institute to the credit of the Column Test Fund which had considerable attention in these pages last month. A great deal more money than that will be needed to get the job done as outlined by Committee 105—Reinforced Concrete Column Investigation, W. A. Slater, Chairman.

New York and Cincinnati have made a fine start (and they are not through) and it remains for other centers to shoulder some of the load. The benefits will be well distributed; the cost, in all justice, should not be borne by a few.

The Institute is very glad to acknowledge cash contributions actually received through Henry C. Turner, in general charge of funding the column tests, as follows:

CONTRIBUTIONS RECEIVED FOR REINFORCED CONCRETE COLUMN TEST FUND

NEW YORK CITY LOCAL COMMITTEE

Received since Nov. 15

Barney-Ahlers Const. Corp.	**250.00**
Ferguson Co., John W., Paterson, N. J.	**250.00**
Goodwin-Gallagher Sand & Gravel Corp.	**100.00**
Nassau Sand & Gravel Co.	**100.00**
O'Brien Brothers, Inc.	**100.00**
Rosoff Sand & Gravel Corp.	**100.00**
Spencer, White & Prentis, Inc.	**100.00**
Stewart & Co., Inc., James	**250.00**
Turner Construction Co.	**100.00**
	$1,350.00

Acknowledged Nov. Journal

Foundation Co., The	$ 100.00
Industrial Engineering Co.	100.00
MacArthur Concrete Pile Corp.	100.00
McCormack Sand Co., Wm. J.	100.00
N. Y. Trap Rock Corp.	100.00
Steers, Inc., Henry	100.00
White Construction Co.	200.00
	800.00
	$2,150.00

CINCINNATI LOCAL COMMITTEE

Received since Nov. 15

Avril Tru-Batch Concrete, Inc.	**25.00**
American Aggregates Corp.	**25.00**
Cincinnati Builders Supply	**15.00**
Dugan, J. N.	**10.00**
Euphrat & Hanly	**25.00**
Ferro Concrete Const. Co.	**100.00**
Fisher-DeVore Const. Co.	**50.00**
Hall Co., T. J.	**20.00**
Hannaford & Sons, Samuel	**25.00**
Hodges Const. Co.	**50.00**
Miller & Son, Wm.	**5.00**
Moores-Coney Corp.	**15.00**
Nutting Co., H. C.	**25.00**
Penker Construction Co.	**25.00**
Van Camp Stone Co.	**25.00**
	$ 440.00

CHICAGO

Turner Construction Co.	**100.00**

BOSTON

Turner Construction Co.	**100.00**

PHILADELPHIA

Turner Construction Co.	**100.00**

SPECIAL CONTRIBUTIONS

Concrete Reinforcing Steel Institute	**$2,500.00**
($2,000 of this contribution acknowledged in Nov. Journal)	
Rail Steel Bar Association	**$2,500.00**
($2,000 of this contribution acknowledged in Nov. Journal)	
Total Cash received to Dec. 11, 1929	**$7,990.00**

MR. LORD ABSTRACTS MR. MCMILLANS BOOK

Book reviewing will soon be an added function of this JOURNAL. *In a separate department, periodical and important bulletin literature be will digested. In this abstract department books in the usual sense will not be included. A greater service in connection with new books, having a major or minor bearing on the thought and work of the field, will be to publish reviews by competent critics. Reviews may properly embrace partial digests along with critical considerations, while criticism has no part whatever in digests or abstracts as comprehended in the Institute's new section of Abstracts considered in some detail elsewhere in these pages. Overriding this distinction there comes to the Editor an abstract of a new book. If the Institute's Abstracting had been in operation a few months earlier, this book, in its original periodical publication (Engineering News Record) would have had most careful consideration. In the meantime it comes out as a book—not, just another text book, just another handbook, but something rather more interesting. It comes at this matter of concrete with a philosophical approach, with some vigor and freshness and sets down the fundamentals in new guise. It is an important, stimulating addition to our literature. Here is an abstract of the book—without so much as a glimmer of what is in the abstractor's mind—a true abstract—in no sense a book "review."*—EDITOR.

"Basic Principles of Concrete Making"—FRANKLIN R. MCMILLAN. McGraw-Hill Book Co., N. Y., 99 pages, $2.00.

Concrete is a mass of inert aggregates held together by hardened paste of portland cement and water. The properties of the paste—strength, watertightness, durability, etc.—determine the properties of the concrete. The quality of paste depends upon characteristics of cement; upon water-cement ratio; and upon time, favorable temperature and continued presence of water to complete hydration of the cement.

A plastic, homogeneous mixture is vital to good concrete making, meaning a mixture in which the constituent materials are evenly distributed in all parts of the mass as finally placed and hardened and one which, when freshly mixed, flows sluggishly but without segregation of water or fine materials from the coarse.

Plasticity depends upon relative quantities of paste and aggregate, upon plasticity of the paste itself, upon grading of aggregates and upon shape and surface characteristics of aggregate particles. The amount of paste must be sufficient completely to fill the voids in the aggregates and to float the aggregate particles. The paste must not be so diluted with water as to be unable to hold the particles in a cohesive mass nor so stiff as to make whole mass difficult to handle and place. Grading of aggregates determines quantity of paste required to float particles and also resistance to ready motion or flow of the aggregate itself. Shape of aggregate particles likewise affects the resistance to flow of the aggregate and of concrete made from it. Surface characteristics affect readiness with which paste surrounds and firmness with which it holds the particles and amount of paste required for plastic concrete.

A portion only of the water used in mixing concrete combines chemically with the cement—if only enough water were used to hydrate cement, concrete would be entirely unplaceable. Re-

mainder of water—commonly the greater part of water used—makes it possible to handle the concrete but is also responsible for most of later difficulties. The greater the amount of uncombined water the less the watertightness of the concrete. Additional curing causes more water to combine with cement.

For ordinary size ranges, size of aggregate has no appreciable effect on strength of concrete, all sizes giving same cement-water strength relations, after proper corrections have been made for water that rises to surface with some mixes. Grading of aggregates does not affect strength, except as it requires variation (to secure plasticity) in water-cement ratio used. Type of aggregate (gravel, crushed stone, slag, etc.) has very little effect on compressive strength of concrete at any given water-cement ratio, absorption of water by aggregate and all other factors being taken into account. For the lower W/C ratios however crushed stone does give greater strength than gravel, for instance, but this is offset by higher W/C ratios required for equal plasticity when crushed stone is used.

Very great increases in strength are secured by continued curing. Curing is a major factor in concrete strength and its importance is not sufficiently realized on job. Age, with continued curing, shows consistent gain in strength following a W/C ratio law at all ages. The superiority of concrete with low W/C ratio over that with high W/C ratio increases with age.

Effect of differences in brands of portland cement on compressive strength are more a matter of *rate of gain* at various periods under favorable curing than of potential strength. At early ages considerable differences are found and a corresponding difference in the position of the W/C ratio strength curve would be expected.

Failure to provide proper curing conditions is even more detrimental to watertightness than to strength of concrete. Lean mixes or thin sections require longer curing than rich mixes or thick sections. For watertight concrete from two to four weeks of curing are required with ordinary materials and mixes. The differences in watertightness due to various cements (including special cements) are far less important than those due to W/C ratio and to curing.

Admixtures generally require use of additional mixing water and result in less watertight concrete, when the consistency (slump) of the concrete is held constant. Powdered admixtures have the same relation to the concrete mass as fine particles of aggregate. Compared on basis of same W/C ratio admixtures increase watertightness, but not enough to offset additional mixing water which their presence requires.

Freezing-and-thawing tests (considered as a measure of permanence in concrete) indicate a close relation between W/C ratios and permanence. Low water-cement ratios increase resistance to weathering, high ratios decrease it, the relation being roughly parallel to W/C ratio strength relation.

Traverse and tensile strength of concrete also follow a definite W/C ratio law. Effect of characteristics of aggregate on these properties is much more marked than on compressive strength. Effect of curing is equally important on these properties.

In practical concrete proportioning grading of aggregates is important. Grading affects strength and other

properties of concrete by varying water required to secure necessary plasticity. Grading giving greater density, requires less water and results in higher strengths. Grading of aggregate is primarily an economic consideration. Well graded materials permit of use of proper W/C ratio to give required quality of paste (and therefore of concrete) without requiring extra cement for plasticity. With a given set of materials best method of determining mix is by trial, using that combination which results in required plasticity (with required quality of paste) at least cost. Excessive sand must be avoided to maintain quality of the paste and also to avoid increases in volume change and air voids and decrease in weight. Excessive sand increases the required *amount* of cement paste and the change in volume appears to vary with the amount of paste in the concrete. A low W/C ratio, however, tends to decrease the volume change, so that rich mixes do not necessarily imply great volume changes. One effect balances the other.

Over-wet concrete is not only bad but it is wasteful, when working to a required W/C ratio. The extra cost of placing prevents use of over-stiff mixes. Greatest promise for improvement in concrete practice is along line of better job curing and better placing methods, the first resulting in more complete combination of the water with the cement (less pores) and the second in elimination of much unnecessary mixing water.

The effect of temperature on curing of concrete must be considered in determining required quality of the paste (W/C ratio) for any exposure. At lower curing temperatures lower W/C ratios are required to give same strength at any given age. Curing period on job should be such as to produce curing equal to that effected by not less than 10 days in damp storage at 70° F. For thin sections or regions of low humidity, etc., this requires 10 days actual curing on job. When conditions are favorable to continued curing after job control is released, this period may be somewhat shortened, but never to less than 5 days moist curing at 60° F.

Recommended minimum W/C ratios are: for extreme exposures—5½ to 6 gal. per sack; for severe climates, ordinary exposures—6 to 6¾ gal. per sack; for moderate climates, ordinary exposures—6¾ to 7½ gal. per sack; for inclosed structural members, 7½ to 8¼ gal. per sack. Lower W/C ratio in each case is for thin walls or slabs or for members with large surface area compared to their volume. Higher W/C ratios are for massive concrete work.

Certain safeguards are essential in construction. Aggregates should meet standard specifications (such as 1928 Joint Code) and should be of durable sound minerals. Rock from deposits showing marked weathering should not be used. All materials must be measured accurately. In measuring water, allowance must be made for the free (surface) water on aggregate particles. Mixing must be adequate to produce homogeneous concrete, with a minimum mixing time of one minute. Segregation must be prevented during transportation of concrete to forms. Placing must likewise avoid segregation and mortar must not be permitted to flow into inaccessible corners—the necessary trouble must be taken to place homogeneous concrete every-

where. Overworking in finishing must be avoided. Finally the man in the office must give the man on the job the necessary latitude, understanding and direction to insure proper results. —ARTHUR R. LORD.

* * *

AT THE AMERICAN ROAD BUILDERS' MEETING

THE AMERICAN ROAD BUILDERS' ASSOCIATION, whose work is of lively interest to many Institute members, will hold its annual convention and exposition at Atlantic City, beginning January 13. It is said that the "show" will use 500 car loads of materials and equipment and the convention the reports of 50 committees. The headquarters hotel is the Ambassador, meetings and exposition at the new Public Auditorium where a room has been graciously set aside by Engineer-Director Charles Upham for the use of members of the American Concrete Institute and other technical societies as a place of get-together.

DISCUSSION

Discussion of papers and reports published this month should be available before March 1, with a view to their publication in the JOURNAL for April, 1930. It is recommended that prospective contributors to discussion make their work available early. Late discussion or the later contributions to discussion which may be too voluminous for a single issue, will be carried over possibly for several months.

BALLOTS FOR OFFICERS AND FOR 1931 CONVENTION PLACE

AS REQUIRED in the By-Laws, ballots were sent to members December 13, listing nominees for offices to become vacant at the 26th annual convention:

President, Duff A. Abrams.

Vice-President, Arthur R. Lord.

Treasurer, Harvey Whipple.

Director, 1st District, R. B. Young.

Director, 2nd District, Cloyd M. Chapman.

Director 6th District, Raymond E. Davis.

Ballots are returnable Feb. 11.

In the same mail postcard ballots were sent giving members an opportunity to express to the Board of Direction a preference as to the 1931 meeting place among the following cities: Chicago, Indianapolis, Kansas City, Louisville, Milwaukee, St. Louis,

* * *

EXTRA BOUND VOLUMES

APPROXIMATELY 300 members asked the Institute to set aside an extra complete set of *Proceedings* pages, beginning with the first JOURNAL issue—these to be bound, uniform with previous volumes, and delivered after the last issue of Volume 26 (June 1930) at a special advance order price of $3.00 each.

No further orders for Vol. 26 can be accepted on this basis. Orders for Vol. 27 can be accepted on a similar basis (exact price to be announced later) up to August 1, 1930 (before the September, 1930 JOURNAL goes to press). This offer is open only to members and subscribers who are receiving the JOURNAL on regular order.

Papers and Reports—In this Issue

Proceedings American Concrete Institute

Vol. 26—Pages 101-112

SOME PERMEABILITY STUDIES OF CONCRETE 101
By F. R. McMILLAN and INGE LYSE

CONCRETE ROADBED ON THE PERE MARQUETTE RAILWAY .. 143
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By HARDY CROSS

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A STUDY OF PORTLAND CEMENT MORTARS HAVING DIATOMACEOUS EARTH AS AN ADMIXTURE 184
By JESSE E. BUCHANAN

A METHOD OF DETERMINING THE CONSTITUENTS OF FRESH CONCRETE 202
By W. M. DUNAGAN

Discussion of the contributions in the following 112 pages is scheduled for publication in the JOURNAL *for April 1930 and should be available not later than March 1.*

Some Permeability Studies Of Concrete

BY F. R. McMILLAN* AND INGE LYSE†

INTRODUCTION

The extensive use of concrete in structures for hydraulic power developments, harbor works, irrigation, water supply, and other construction fields has brought to the attention of engineers that among the properties of concrete, watertightness may be of even greater importance than compressive strength. The importance of impermeability is not so much a matter of confining the water to its proper channels, for almost any concrete carefully made and cured will prevent serious loss of water through percolation. The real need for watertightness is to prevent the disintegration which results from the freezing of saturated porous concrete or that slow breaking down through the solution of essential ingredients. It needs only a brief survey of structures which have been exposed to severe climatic conditions to appreciate how important is the destructive effect of frost where the concrete is readily permeable. Likewise, there are everywhere examples of structures where unsightly deposits of calcium carbonate bear evidence of water percolating through some defective area or seeping along a day's work plane.

Typical examples of concrete structures in service are shown in Fig. 1. The picture at the left shows a wall in which the coping and upper portion are breaking away due to the freezing of contained moisture. The picture at the right shows a portion of a dam with heavy deposits of calcium carbonate at several fill planes and at intermediate points where water has found its way through defective spots. These illustrations show forcefully the need for study of watertightness so that concrete can be proportioned and placed to avoid these difficulties.

The tests reported in this paper were undertaken at the Research Laboratory of the Portland Cement Association, for the special purpose of studying the watertightness of concrete mix-

*Director of Research, Portland Cement Association, Chicago.
†Assistant Engineer, Portland Cement Association, Chicago.

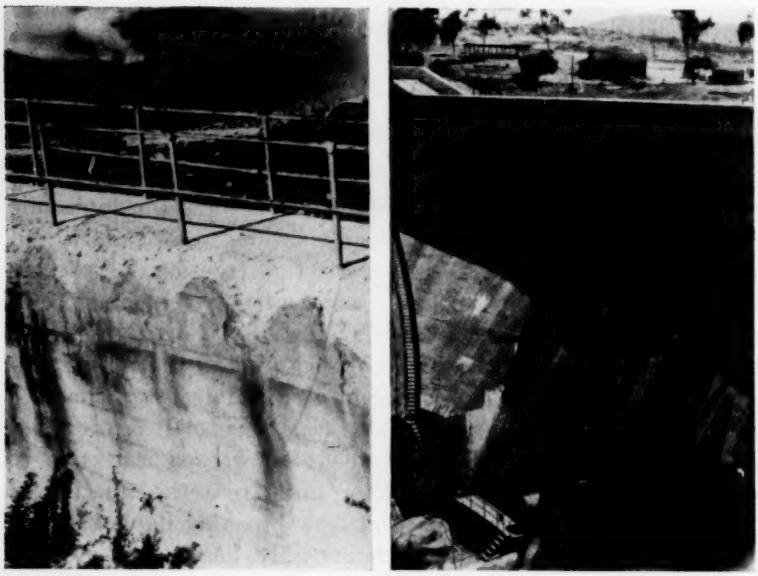


FIG. 1—EXAMPLES OF CONCRETE IN SERVICE.

LEFT, PORTION OF WALL SHOWING DISINTEGRATION AT TOP DUE TO FREEZING OF POROUS CONCRETE.

RIGHT, PORTION OF DAM SHOWING INCRUSTATION DUE TO SEEPAGE ALONG JOINTS.

tures as part of a general investigation covering the factors affecting the durability of concrete. This particular group of tests has been under way for the last year. While it is by no means complete, even in the limited field to which the tests are restricted, the results to date are of sufficient interest and importance to warrant their publication at this time.

In view of the attention which is now being given to permeability tests of concrete, some space in this report is devoted to a description of the apparatus used and the reasons for its adoption.

PERMEABILITY APPARATUS

The type of apparatus used in these tests was determined largely by certain limitations imposed by the magnitude of the proposed program. The large number of variables to be investigated made it a matter of first concern that the apparatus be easily duplicated, and that it be of such design that specimens

could be inserted and removed with a minimum loss of time. Another feature of the program which influenced the type of apparatus was the intention to include tests of concrete specimens cut from larger masses of concrete. This latter feature was responsible for the use of disc specimens 6 in. in diameter, for it was desired to test some specimens cut from standard 6 by 12-in. cylinders so that the permeability data could be very closely related to the wealth of information on concrete mixtures, which is largely recorded in terms of compression tests on specimens of this size.

In addition to the above, there was imposed the further arbitrary limitation that the records must be based on the measurement of water actually passing through the concrete, rather than on the amount of water going into the specimen, as has sometimes been done. The purpose of this was twofold: first, to give an opportunity to study the solvent action of the percolating water; and second, it seemed like a more convincing method of comparing the watertightness of different mixtures or the effect of different conditions of test.

To meet these several requirements, the apparatus shown in Fig. 2 was designed. This permits the use of discs 6 in. in diameter and of any desired thickness up to 4 in. The water passing through is caught in a glass funnel 4 in. in diameter held against the center portion of the bottom surface by means of springs. The funnel discharges either directly or through a rubber hose into a glass graduate which is loosely stoppered to prevent loss through evaporation.

The apparatus is easily duplicated as it is an assembly of standard pipe fittings. The only piece which requires special machining is the 6-in. extra heavy duty wrought iron pipe nipple. This is machined at the threaded end to a diameter of $6\frac{1}{8}$ in., leaving a $\frac{1}{2}$-in. shoulder upon which the specimen is seated. Forty-four such units are installed and have been in almost constant use in this series of tests for 12 months. Some extra units are fitted with smaller pipe to permit the testing of discs 3 and 4 in. in diameter.

A number of individual units are connected at 14-in. intervals on a horizontal pipe 4 or 6 in. in diameter which acts as a water reservoir in which the pressure is supplied by compressed air.

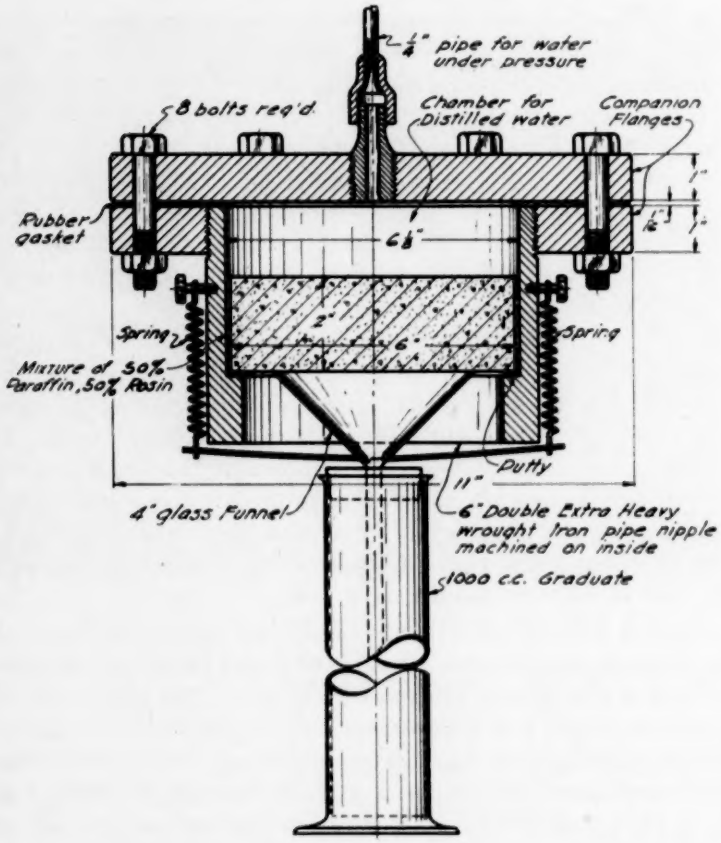


FIG. 2—APPARATUS FOR MEASURING PERMEABILITY OF CONCRETE. (SCALE 1" = 4")

There are five groups of units which can be carried simultaneously at the same pressure or at a different pressure on each group. The pressure on each line is held constant by a regulating valve and is recorded on a recording gage. Any unit can be disconnected from the line at any time without interfering with the operation of the other units.

Before placing the specimen in the apparatus, a thin layer of putty is placed on the shoulder. The specimen is then pressed against the putty so that a tight joint and uniform bearing is

obtained. The small annular space between the specimen and the pipe is then carefully filled with a bonding material consisting of a molten mixture of equal parts of paraffin and rosin. This gives a satisfactory bond for pressures up to 140 lb. per sq. in. which is the maximum used with the apparatus. After the specimens have been tested, the top flange is removed, the unit inverted and heated on the outside by means of a gas flame until the bonding material melts and the specimen drops out.

THE RESISTANCE OF CONCRETE TO PENETRATION OF WATER

The study of watertightness of concrete will be greatly simplified if concrete is thought of as an aggregate mass thoroughly incorporated in a cement-water paste. If the aggregate particles are impervious, obviously any water which finds its way through the mass must pass either through the cement paste or through openings due to incomplete filling of the space with paste. Under this simple conception of concrete, it can be seen that there are in effect only three requirements for watertightness, namely, (1) impermeable aggregates, (2) a cement-water paste which, when hardened, will be impermeable, and (3) a mixture such that the paste completely fills the spaces between the aggregate particles. A brief consideration of these three requirements will clarify the discussions of the test data which follow.

So far as the permeability of the aggregate is concerned, the problem of producing watertight concrete need not be difficult. Most of the common aggregate materials are sufficiently watertight to resist the destructive effects of frost almost indefinitely. These are so well distributed and their behavior so well known that it is the unusual case where difficulty is encountered. Among the porous aggregates that do not resist weathering well when exposed to dampness and frost, are many sandstones and some limestones. These, as well as other porous materials, should be avoided where great watertightness or resistance to weathering is required. In the tests reported herein, the fine and coarse aggregate used were calcareous sand and gravel which have proved very satisfactory in severe exposure. So far as these tests are concerned, the aggregates can be considered impermeable. The study of porous aggregates is reserved for a later part of the investigation.

The other requirements of watertightness, (2) and (3) above, which may be referred to briefly as the quality and distribution of the paste, involve all those factors which constitute the science and art of concrete making and placing.

The quality of the paste has been found to depend on three factors: (a) the characteristics of the cement, (b) the proportions of the cement and water used, and (c) the extent to which the chemical reaction between the water and cement has been allowed to progress. Similarly, the distribution of the paste in a mixture depends upon the amount and consistency of the paste and upon the size, grading, proportions, and other characteristics of the aggregate, and upon the methods of placing. Many of these factors were studied in the permeability tests which have been published previously, but they were considered as independent variables so that the results of such tests are only indirectly applicable to the design of concrete mixtures. Studies of the other properties of concrete have shown that these are not independent variables, but are only various means through which the quality and distribution of the paste can be controlled. In this group of studies, principal attention has been given to the factors which govern the quality of the paste, for the importance of complete incorporation of the aggregate in the paste and the methods through which it must be secured will be recognized at once by the thoughtful student.

PREPARATION, THICKNESS, AND AGE OF TEST SPECIMEN

At the outset of this investigation it was hoped that a large proportion of the specimens could be cut from 6 by 12-in. cylinders. The series expanded so rapidly, however, that it was found impracticable to cut so large a number of specimens. A small series of tests were, therefore, carried out to determine if any considerable error would be involved in using molded discs 2 in. thick in place of the 2-in. disc cut from 6 by 12-in. cylinders with a carborundum saw. This series of tests showed that the molded discs gave practically identical results with those cut from the larger specimens when the surface films were removed from the molded specimens. To remove the surface film the practice was established of grinding the surfaces on an emery stone and brushing with a wire brush before placing under test. A layer

$\frac{1}{16}$ to $\frac{1}{8}$ in. thick was thus removed from the top and bottom surfaces of each disc.

The thickness of the specimen used has generally been 2 in. which gives satisfactory results where the aggregate does not exceed $\frac{3}{4}$ in. in maximum size. For specimens of this thickness, pressures of 40 to 80 lb. per sq. in. were used, depending upon the character of concrete at the time it was subjected to the test.

For the study of mortar specimens, a thickness of 1 in. has been adopted which seems to be satisfactory for pressures up to 20 lb. per sq. in. Since the specimens all have the same diameter, the pressure must be varied to suit the thickness and character of concrete so that a specimen will not fail due to bending stresses. The resisting moment being a function of the square of the thickness of the specimen, 20 lb. per sq. in. pressure on a 1-in. thickness gives the same bending stress as 80 lb. per sq. in. on a 2-in. thickness.

In the early work of the series the attempt was made to use the standard 28-day moist curing at 70° F. for all specimens for test. It was found however, that for all the mixes which are within the range of commercial practicability, the leakage was so small that reliable comparisons could not be made between the different variables. In fact, for most mixes tried, it was impossible to get any measurable quantity of water through the 2-in. specimens even when pressures up to 120 lb. per sq. in. were used.

It was necessary, therefore, to adopt a shorter moist curing period as a basis of comparison. For most of the tests, specimens were cured moist for an initial period of 3 or 7 days which was followed by various periods of air curing before submission to water pressure. The moist curing was always done at the temperature of 70° F. and the air curing in a room in which the temperature was maintained at approximately 70° F. The effect of different curing periods on the permeability forms an important part of the investigation which will be brought out in subsequent paragraphs.

The fact that most of the specimens were submitted to the test at an early age introduced the element of additional curing due to the presence of moisture in the specimen while the test was in progress. This is also covered in the discussion of the test results to follow.

GENERAL PHENOMENA OF TESTS

In all the tests it was found that for those specimens which showed leakage the flow reduced as the test continued, the rate of change depending upon the quality of the concrete and the age of the specimen.

Fig. 3 shows a typical leakage-time curve for concrete specimens made from two different cements. Each curve represents an average of 3 specimens and the individual plotted points show

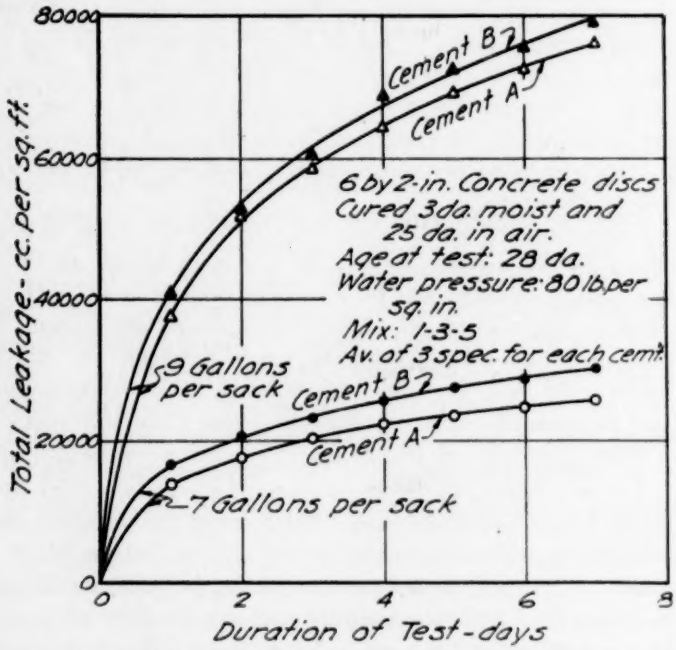


FIG. 3—TYPICAL LEAKAGE CURVES FOR CONCRETE SPECIMENS.

the total flow through the specimen at a given period, beginning from the time the test was started at age of 28 days. This diagram brings out many of the characteristics of permeability tests, particularly the reduction in flow with time. On this point, it will be noted that of the entire flow in a period of 7 days, about half occurred in the first day. It will be observed also that after about the first 3 days the flow continues at a more nearly constant rate.

In this first part of the investigation there were so many conditions for which information was desired that tests of only short duration were employed. It can be seen from Fig. 3 that unless the test is carried to the point where the rate of leakage remains practically constant, it is very important that comparisons of different variables be made at some definite time. In many of the tests discussed herein, the comparisons are made on the basis of leakage in the first 48 hrs. That such comparisons at early periods are entirely justified can be seen from Fig. 4 where, for example, the variation in leakage with water-cement ratio is shown for several different periods. These curves show the results from a group of tests plotted to bring out this particular feature. It can be seen from this figure that in comparing the different water ratios, similar conclusions would be reached no matter which of the several periods was used.

In the illustration used in Fig. 4, another effect of this time factor is introduced, that is, the effect of additional curing during the prolongation of the test. The specimens used in the tests of Fig. 4 were exposed to the water pressure only 3 days after making so that with each succeeding day on the line, the specimen gains in watertightness. This point was referred to in the previous section and will be discussed further under "Effect of Duration of Test on Permeability."

Another characteristic of the permeability test is its extreme sensitiveness. In this respect, it is not at all comparable with compression tests where, for example, strengths commonly fall within a range of 10 to 20% in a group of specimens. In these tests for watertightness, the specimens were small, making the factors of placing and curing, which materially affect the permeability, very important. A very slight honeycomb spot or water channel, which would have no appreciable effect on compressive strength, can affect the leakage to a very important degree. Differences in temperature of the water and minor variations in pressure also contributed to the differences in results between individual specimens, which sometimes were as much as 50 to 75%. It was because of these difficulties that the practice was followed of making companion specimens on different days so that error in making and testing would be minified. The use of a number of tests of a kind is much more important in the case of permea-

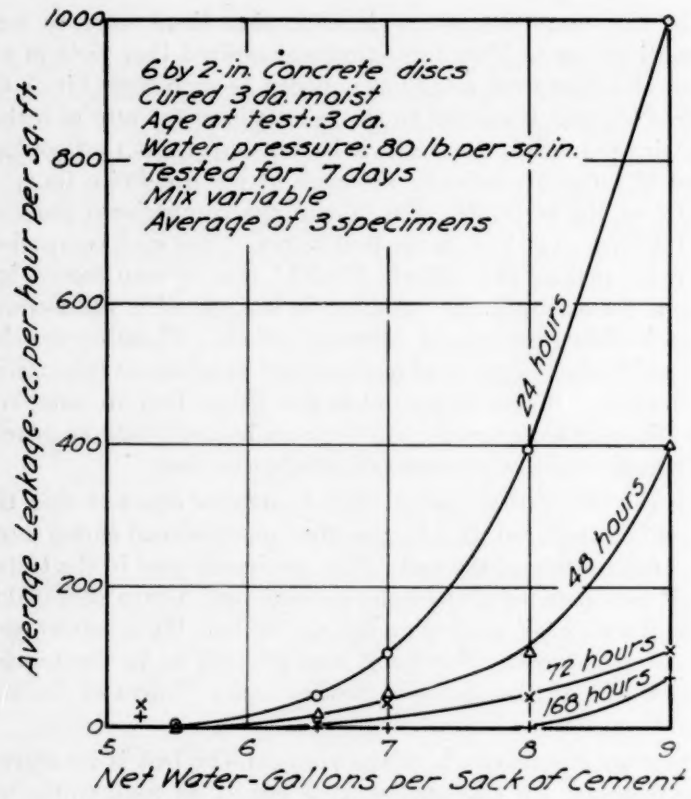


FIG. 4—AVERAGE LEAKAGE FOR DIFFERENT PERIODS OF TEST.

bility tests than for compression tests. In these tests, from 3 to 5 specimens of a kind were always made. It will be seen from the regularity of the plotted data that the results are, in general, very reliable.

It was frequently observed in the tests in this series that when the space between the particles was not completely filled with paste, definite leakage in the form of small streams of water was detected. Such condition could result either from a general deficiency in the quantity of paste or from the failure to properly compact the concrete in the form. A similar condition was also found in the case of overwet mixes where small channels seemed

to have been formed by the excess water rising to the top during the period immediately following placing. In these tests, mixes in which the paste was insufficient in quantity to thoroughly fill the space between aggregate particles were definitely avoided, for it was known in advance that they could not be watertight. Leakage in the form of pin-like streams, therefore, indicated either a wet mix or an incomplete consolidation of the concrete in molding.

The possible differences in permeability resulting from differences in placing methods are brought out in the curves of Fig. 5, which show the results of tests on specimens molded by rodding

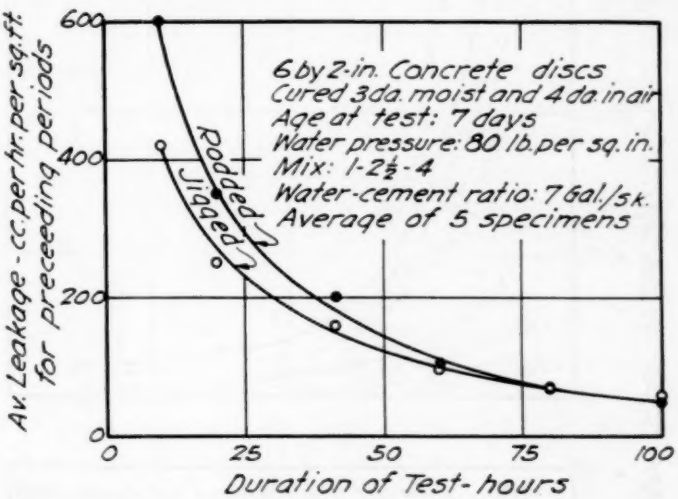


FIG. 5—EFFECT OF METHOD OF MOLDING ON PERMEABILITY OF CONCRETE.

in the usual way and similar specimens jigged 10 times on a standard flow table with ½-in drops. These results are the average of 5 tests of 2-in. discs under a pressure of 80 lb. per sq. in. The mix was 1:2½:4 and the water-cement ratio 7 gal. per sack of cement. The tests were made at 7 days after 3 days moist curing and 4 days in dry air. The jigged specimens show greater watertightness during the first two days, but thereafter, the leakage is about the same. By comparing these data with those given in Fig. 4, it can be seen that the difference in leakage

between the two methods of placing after 24 hrs. under test is comparable with that between 7½ and 7¾ gal. per sack in the other series at the same period of test. This comparison is of interest only in showing the relative importance of these differ-

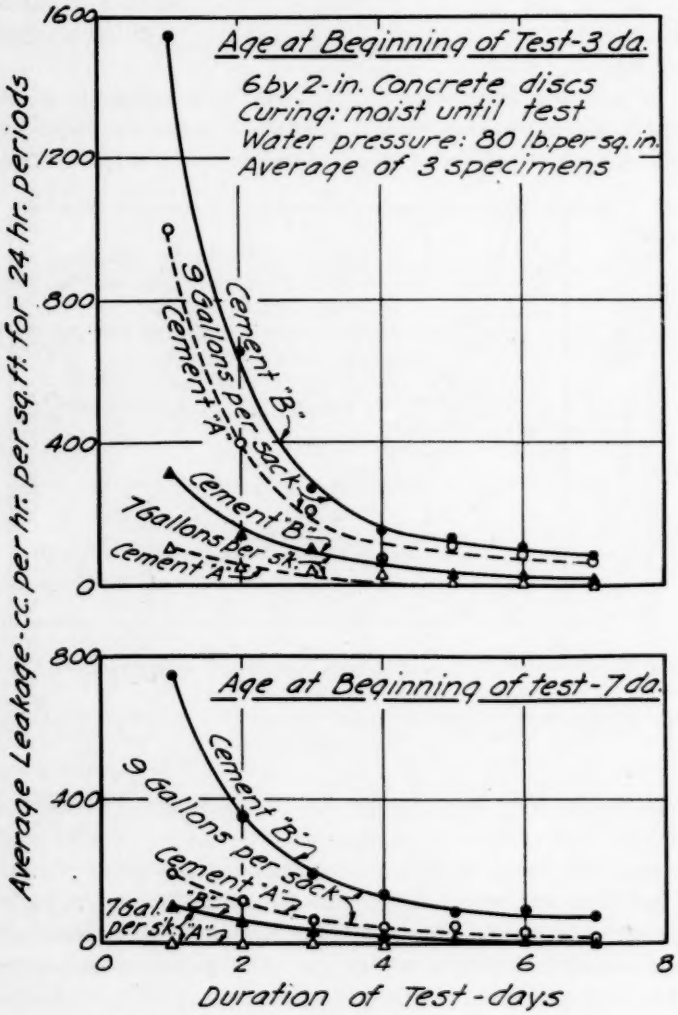


FIG. 6—EFFECT OF DURATION OF TEST ON THE PERMEABILITY OF MOIST-CURED CONCRETE.

ences. A direct comparison cannot be made because the age and curing condition are not identical.

EFFECT OF DURATION OF TEST ON PERMEABILITY

It has been mentioned previously that the leakage decreased with the duration of the test. The leakage the first few days was much larger than the leakage after 4 or 5 days. Many specimens sealed themselves entirely within a rather short period of test. To a certain extent this was to have been expected from the continued hydration during the early period of moist curing which was found to be of extreme importance. The time the specimens were under pressure was actually a moist curing period and at least part of the decrease in leakage found by increased length of test can be accounted for in this manner.

A clear illustration of the effect of duration of test on the flow through the specimen is given in Fig. 6. The specimens in this series which represent two different cements and two different water-cement ratios were kept under a constant pressure of 80 lb. per sq. in. for 7 days, the leakage being measured every 24 hours. The upper diagram shows the results from specimens placed under test at age of 3 days while the lower gives the results for specimens 7 days old when tested. The specimens were all cured moist at 70° F. until the day the test was started. The curves show a very good uniformity in the leakage from day to day. The leakage decreased very rapidly the first 3 to 4 days of test and then approached a more nearly constant decrease. This was especially true for specimens with high initial leakage. Specimens leaking only slightly at the beginning of the test showed a more uniform decrease throughout the duration of test.

Fig. 7 shows leakage curves for a relatively long test. The specimens were 6 by 2-in. concrete discs tested at 80 lb. per sq. in. for 25 days. The age at the beginning of the test was 28 days; curing was 7 days moist and 21 days in air, all at approximately 70° F. The curves for the fairly permeable concrete show a sharp decline in leakage for the first few days of test while those for the only slightly permeable concrete show a gradual decrease until entirely sealed.

In order to study the phenomenon of duration of test further, Fig. 8 is presented. In these tests two specimens for each water-

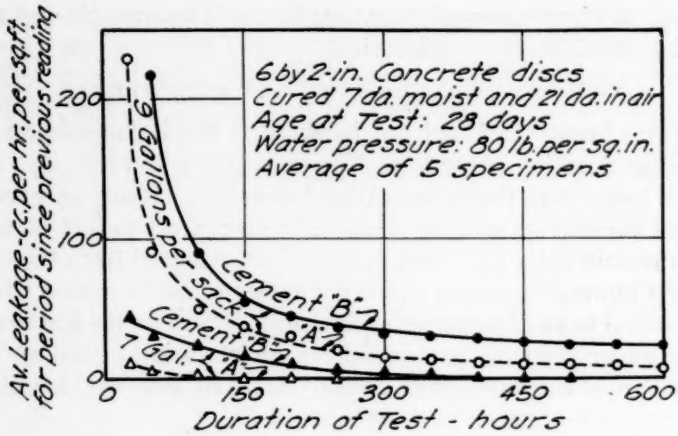


FIG. 7—LONG-TIME PERMEABILITY TEST OF CONCRETE SPECIMENS.

cement ratio were made from a single batch of concrete. They were cured moist at 70° F. until day of test. One specimen was placed under test at the age of 3 days and the companion specimen at 7 days. The duration of test was in both cases 7 days. The plotted points represent the average of 3 specimens which were made on 3 different days.

If the decrease in leakage with duration of test were due entirely to greater length of moist curing, the first day's leakage of the 7-day specimens would be equal to the 4th day's leakage of the 3-day specimens. This is seen not to be the case. The 7-day specimens showed a considerably higher leakage the first few days of test than did the 3-day specimens which were of the same age but were longer under test. After about 3 days under test, however, the leakage of the 7-day specimens approached very closely the leakage of the 3-day specimens for the corresponding total age. This seems to indicate that the decrease in leakage with time was not due entirely to the effect of additional moist curing, but partly to the filling of the pores with silt or other impurities in the water, or in some other manner.

In a few tests distilled water was used instead of the ordinary city supply water. On account of the oxidation of the steel equipment resulting from contact with the air used for producing the pressure, the distilled water very soon became impure. The

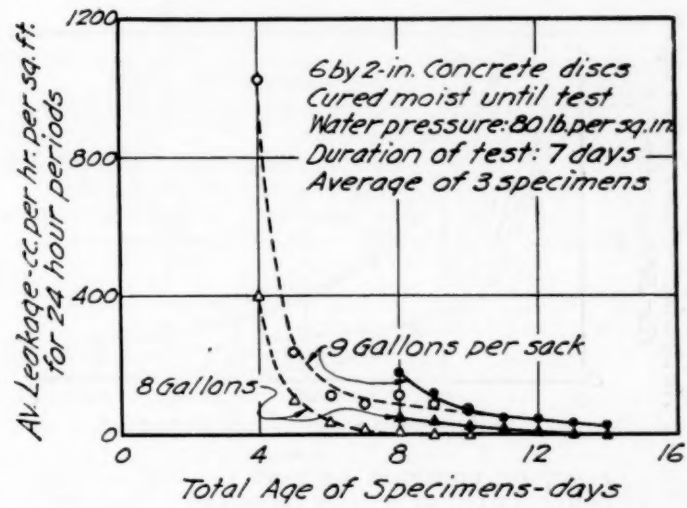


FIG. 8—EFFECT OF DURATION OF TEST ON PERMEABILITY OF CONCRETE TESTED AT 3 AND 7 DAYS.

results from these tests, therefore, were quite inconclusive as they showed no large difference between distilled water and that from the ordinary city supply.

In order to get rid of some of the silt and other impurities in the water, a filtration and settling system was installed between the city supply and the system of permeability apparatus. The amount of iron oxide formed inside the pipes and lines in the permeability apparatus was not eliminated by this procedure, so very little improvement was obtained by the use of the filter. It was noted that specimens which had been subjected to test for a long period showed a coating of iron oxide and other impurities on the top surface indicating that a considerable amount of sealing might have taken place in the specimens during the period of test. If filtering is to be required, the filters should be placed in each unit so that impurities are removed just before the water comes in contact with the specimens. Some further tests are planned to study this sealing effect, also to compare the action of acid and alkaline waters of different concentrations. In these it will be necessary to control the impurities in the water. The city water contains some bicarbonate which is converted to calcium carbonate on contact with the concrete. A precipita-

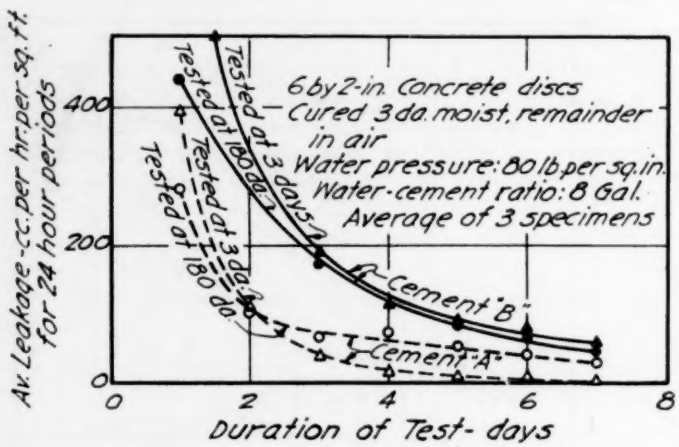


FIG. 9—EFFECT OF DURATION OF TEST ON PERMEABILITY OF CONCRETE CURED 3 DAYS MOIST AND TESTED AT DIFFERENT AGES.

tion of calcium carbonate within the pores was doubtless responsible for some of the sealing effect.

A decrease in leakage with duration of test was also found for specimens cured in air for a long time before test. This decrease was, however, not quite so large as that found at earlier ages. This is shown in Fig. 9 where the similarity of the curves for the tests at the different periods indicates that no fundamental difference exists. The heavy curves in this figure represent specimens containing Cement B, while the light curves represent the leakage of identical specimens made from Cement A. The curves indicate that the 177 days of air curing following the 3 days moist curing did not change the resistance of the specimens materially, neither in the total leakage nor in the rate of change with duration of the test.

Aside from the interesting comparison just made with respect to the curves in Fig. 9, the data are of interest also in reference to the effect of air curing on permeability. In fact, the really significant conclusion to be drawn from these data is that 177 days additional age in air has added nothing to the watertightness of this concrete. The importance of this will be further brought out in the discussion of curing.

EFFECT OF WATER-CEMENT RATIO

In the general analysis of the resistance of concrete to the penetration of water, in a preceding article, the three factors which determine the quality of the hardened cement-water paste were given as follows:

(a) The characteristics of the cement

(b) The proportion of cement and water—i. e., water-cement ratio

(c) The extent to which the chemical combination of cement and water has progressed, i. e., curing.

These three factors formed the principal subjects of inquiry in this first series of tests because it was recognized that in the last analysis the quality of the paste must be the essential element through which watertightness is controlled. The other factors, the permeability of the aggregates, and the manner of placing to obtain a mass free from honeycomb, water channels, or air voids, are so obvious in their effect that their consideration has been deferred to a later part of the investigation.

Because of the fundamental nature of the three factors, character of the cement, water-cement ratio, and curing, they cannot be wholly separated; in fact, in the data presented so far (Fig. 3 to 9) each has received incidental attention. In this and the two succeeding sections, however, the data will be studied with particular reference to these factors.

The primary influence of the water-cement ratio on the permeability of concrete and mortar is brought out very clearly by the data in Fig. 10 and 11. The former shows some typical water-cement ratio-leakage curves for concrete and the latter, similar curves for mortar. In both of these groups of tests two cements were used and different curing periods employed. Owing to the difference in thickness of discs, the tests were carried out at different pressures.

In the concrete tests of Fig. 10, the specimens were 6 by 2-in. discs tested under a pressure of 80 lb. per sq. in. The tests on one set of the specimens were started at age of 3 days, while the tests on another set started at 7 days, the specimens all having been moist cured at 70° F. until test. The leakage shown is the average in cc. per hr. per sq. ft. of area for the total time of test

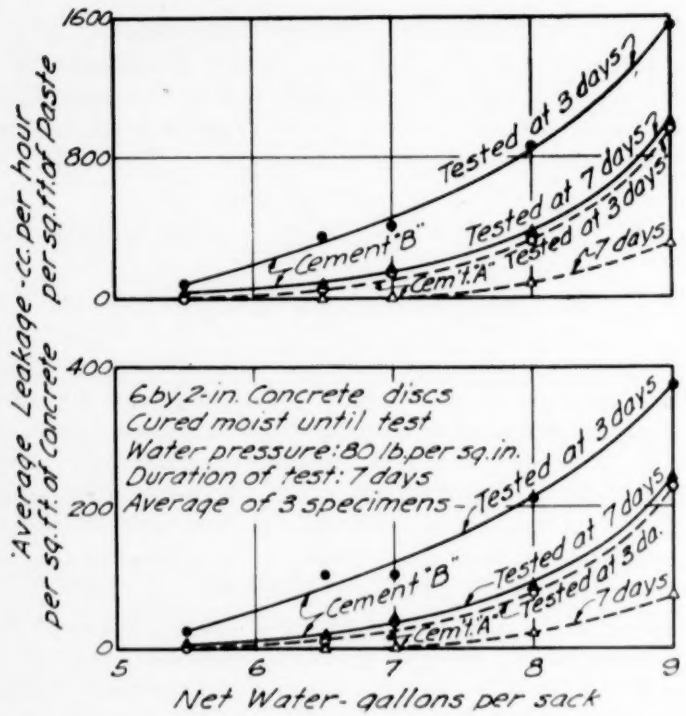


FIG. 10—EFFECT OF WATER-CEMENT RATIO ON PERMEABILITY OF CONCRETE.

which, in this case, was 7 days. The leakage, of course, varied with the time under test as pointed out in the discussion of Fig. 3, 4, 6, and 7. In the lower diagram this leakage is expressed in terms of the unit area of the concrete. In the upper diagram, this has been reduced to the equivalent leakage per square foot of paste. This reduction is accomplished by dividing the leakage per unit area of concrete by the percentage of paste volume in the concrete. This assumes that the paste is uniformly distributed throughout the mass so that on any section the area of paste bears the same relation to the total area as the paste volume does to the total volume.

It will be noted in Fig. 10 that the leakage increased very rapidly with increase in the water-cement ratio and at an increas-

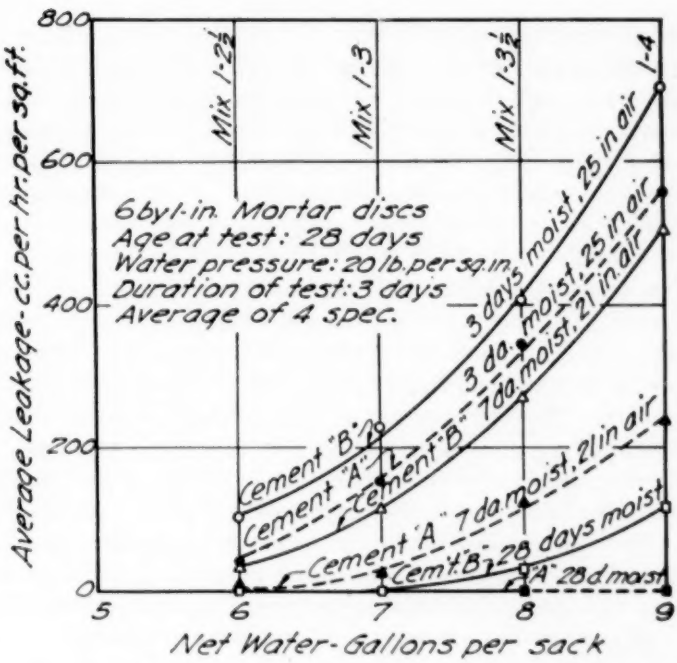


FIG. 11—EFFECT OF WATER-CEMENT RATIO ON PERMEABILITY OF MORTARS.

ing rate as the water-cement ratio became larger. For example, the increase caused by raising the water-cement ratio from 8 to 9 gal. was larger than that caused by raising the water-cement ratio from 7 to 8 gal. It was also seen that the leakage of specimens tested at 3 days was considerably larger than for specimens tested at 7 days. This will be discussed in the next section. The similarity of all curves shows the consistent relation between the water-cement ratio and leakage for different cements as well as for different ages.

For a direct comparison between different mixtures of the effect of water-cement ratio on the watertightness of the paste, the basis used in the upper diagram of Fig. 10 is the more exact, for it takes into account the relative quantity of the paste. For two mixes with exactly the same quality of paste, the leakage per unit area of concrete would not be exactly the same unless the

paste volumes were also the same. The paste volume or quantity of paste is a function of the aggregate proportions, grading, and consistency. In the tests forming the basis of Fig. 10 two different consistencies were used for each of the three mixes—1:2:3, 1:2½:4, and 1:2½:5, to obtain the several water-cement ratios. This corresponded to a variation in paste volumes from 22.5% to 28.8%. The points on the upper curve were obtained by dividing the leakages shown in the lower diagram by the percentages representing the paste volumes.

It will be seen that the curves in the upper diagram are very similar to those in the lower diagram so that the effect of varying the quantity of mixing water can be determined equally well from either set. This is because the range in paste volumes is quite narrow and varies systematically for this group of mixes. In comparing mixes of widely different character, as for example, mortar and concrete, a better comparison might be made on the basis of paste area instead of total area. Because the paste volumes do not cover a wide range in most of the groups of tests in this study, the practice of presenting the data in terms of the unit area of the concrete has been followed generally in this report. Some further consideration of this subject is given in the section "Effect of Direction of Pressure and Amount of Paste."

In the mortar tests shown in Fig. 11 the specimens were 6 by 1-in. discs, tested under a pressure of 20 lb. per sq. in. The tests in this case were all started when the specimens were 28 days old. The specimens, however, were subjected to different periods of moist curing during this period. The tests were continued for a period of 3 days only, and the leakage shown in Fig. 11 is the average in cc. per hr. per sq. ft. for this period. It will be noted that the curves for the mortar specimens are similar in all respects to those for the concrete specimens in Fig. 10. The leakage, however, for any given water ratio for the two sets of tests cannot be directly compared because of the differences in curing, thickness of specimen, and pressure employed. The purpose of this study primarily was to bring out the effect on the leakage of changing the water-cement ratio. It is only by carrying the tests through a considerable range of curing conditions, size of specimen, and pressure, and by using different cements that the

primary effect of the water-cement ratio on the watertightness can be established. That this primary effect is established can be seen from a study of the curves in Fig. 10 and 11 which clearly show how changes in the quality of paste brought about by changes in the proportion of water to cement affect the leakage of the concrete or mortar in a regular and systematic manner.

EFFECT OF CURING AND AGE

The important effect of the length of moist curing and age of specimens on the leakage has already been brought out in the discussion of the preceding section. For example, in Fig. 10 the average leakage through a specimen, which was placed under test at 3 days, was several times as great as that through a similar specimen that had been allowed to cure 7 days before being placed under pressure. Also, in Fig. 11, for specimens placed under test at 28 days, a very decided improvement in watertightness was shown for the specimens cured 7 days moist as compared with those cured only 3 days moist, and a far greater advantage for those cured the entire time in a moist atmosphere.

In Fig. 12 the data used in plotting Fig. 11 are replotted to bring out more prominently the effect of length of moist curing. In this figure separate curves representing the variation in leakage with number of days in the moist room are shown for each water ratio and each of the two cements. All the specimens in this group being the same age (28 days) at the time they were placed under pressure, the difference in leakage indicated is due entirely to the effect of the additional hydration resulting from the extra time in the moist room. Aside from the differences shown by the two cements, which will be discussed in the following section, the interesting feature of this set of curves is the great similarity between the curves for the different quantities of mixing water. As pointed out in the discussion of the water-cement ratio, the watertightness of the concrete is primarily a function of the quality of the paste as indicated by the proportion of water to cement and extent to which hydration is carried out. These two factors act independently of each other. That is, increased watertightness can be obtained either by additional curing, regardless of the quantity of mixing water used, or for any given curing condition by reducing the quantity of mixing water.

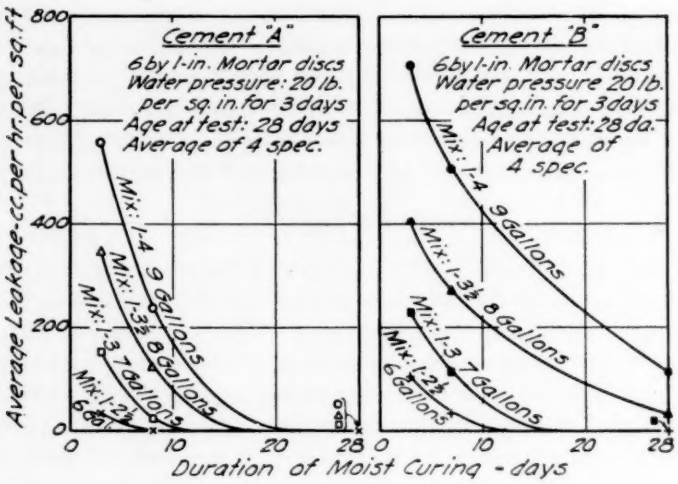


FIG. 12—EFFECT OF DURATION OF MOIST CURING ON PERMEABILITY OF MORTARS.

In Fig. 13 and 14, two other groups of tests are shown in which the effects of age and continued moist curing are also brought out quite sharply. Fig. 13 is based on mortar tests on 1-in. discs tested under a pressure of 20 lb. per sq. in., and Fig. 14 on concrete discs 2 in. thick tested under 40 lb. per sq. in. In both cases the specimens were under test for a period of 48 hr. and the leakage shown is the average for the period in cc. per hr. per sq. ft.

As pointed out in an earlier section, it was found necessary to test the specimens in this investigation at early ages, because, with the mixes ordinarily used in concrete practice, it was found that specimens were so nearly watertight if cured for the more usual period of 28 days that no important results could be obtained from the tests. When tested at the early age, the effect of continued moist curing in increasing the watertightness tends to reduce the leakage with duration of the tests more rapidly than would be the case if the specimens had been tested at the advanced period. This has been brought out in the discussion of Fig. 8 and must be kept in mind in a study of Fig. 13 and 14.

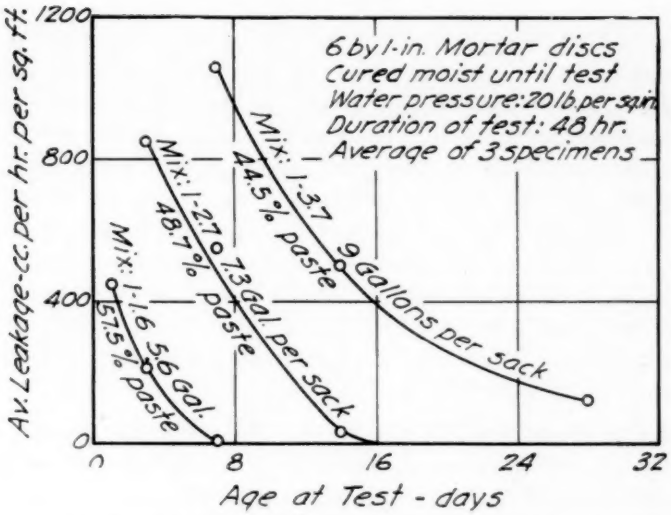


FIG. 13—EFFECT OF DURATION OF MOIST CURING ON PERMEABILITY OF MORTARS.

In Fig. 13, as in those previously discussed, both the effect of curing and water ratio are brought out. In this figure are

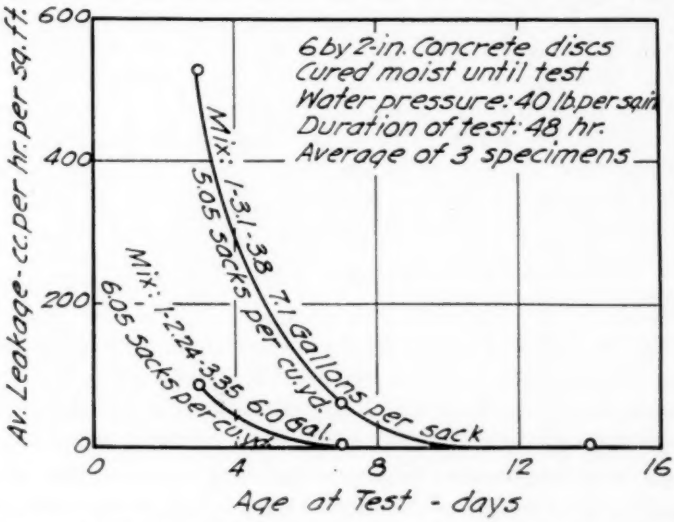


FIG. 14—EFFECT OF DURATION OF MOIST CURING ON PERMEABILITY OF CONCRETE.

presented the results of tests on mortars of 3 different water-cement ratios, a separate curve being shown for each mortar. In these tests, three separate specimens were made from a single batch which were cured in the moist room at 70° F. until day of test. For the 5.6-gal. mixture, one specimen from each batch was tested at one day, one specimen at 3 days, and the third at 7 days. A similar procedure was followed with the other two mortars except that the specimens were cured longer before test owing to their lower strength. In all cases three specimens of a kind were made upon which the average leakages shown are based. The tests in each case were conducted for 48 hr. under a pressure of 20 lb. per sq. in.

In this figure the importance of the water-cement ratio as a primary factor in the watertightness of the mix is indicated by the regularity of the curves for the 3 separate mixes. The effect of the longer period of moist curing is clearly shown by the uniform reduction in leakage with each increase in the curing period. The significance of this increase in watertightness with the period of moist curing can be seen by comparing the time required to produce a given watertightness with the various mixtures. For example, in the richer mix, a curing period of 3 days in the moist room gave a leakage of 200 cc. per hr. per sq. ft. With the two leaner mixes the same watertightness was obtained by increasing the moist curing period to 11 and 22 days, respectively. In these, as in all the tests reported in this paper, all curing was done at approximately 70° F. Tests of strength have shown that lower temperatures produce lower strengths. It is to be expected therefore that lower temperatures would also result in lower resistance to the penetration of water, for a given length of curing.

The curves in Fig. 14 for concrete discs are generally similar to those for the mortar discs in Fig. 13, indicating a like rapid increase in watertightness with continued moist curing and a similar relation for different quantities of mixing water.

The very important effect of continued moist curing on the watertightness as brought out in Fig. 13 and 14 was the most significant feature of this entire investigation. Before the tests were started, it was recognized that the building up of the internal

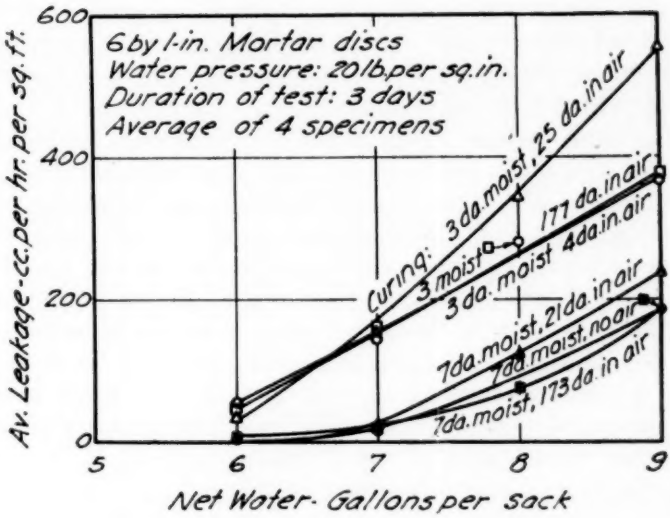

6 by 1-in. Mortar discs
Water pressure: 20 lb. per sq. in.
Duration of test: 3 days
Average of 4 specimens
Curing: 3 moist
3 da. moist, 25 da. in air
3 da. moist 177 da. in air
3 da. moist 4 da. in air
7 da. moist, 21 da. in air
7 da. moist, no air
7 da. moist, 173 da. in air
Av. Leakage - cc. per hr. per sq. ft.
600
400
200
0
5
6
7
8
9
Net Water - Gallons per Sack

FIG. 15—EFFECT OF DURATION OF AIR CURING ON PERMEABILITY OF MORTARS.

structure of concrete through increased hydration must have a major influence on the resistance to the passage of water, and such tests as were available supported this belief. But results such as have been pointed out were hardly expected.

The almost negative value of extended curing in the ordinary air of the laboratory was also quite surprising. This was brought out from the tests recorded in Fig. 15 which show the water ratio-leakage curves for 2 groups of specimens cured, respectively, 3 and 7 days in the moist air and at varying lengths of time in the ordinary air of the laboratory. Periods of dry air curing were 4, 25, and 177 days, giving ages at test of 7, 28, and 180 days. It will be seen that except for minor differences in the case of the 28-day tests for the 3 days moist curing, specimens of each group gave substantially the same leakage for the same water ratio. In other words, the 180-day specimens cured only 3 days in the moist air gave no higher resistance to the penetration of water than the 7-day specimens also cured for the first 3 days in moist air. Similarly, with tests for 7 days moist curing, no advantage in watertightness was shown for the extra 173 days or

21 days air curing. This is the more remarkable when it is compared with the data in Fig. 12 where it is shown that a very pronounced increase in watertightness results from a few additional days of moist air curing.

The higher leakage shown in Fig. 15 for the tests of specimens cured 3 days moist and 21 days in air, in comparison with 7 and 180-day tests in the same group, seems to be significant only of the sensitive character of the permeability tests. That this is true is indicated by the fact that if the readings for 3 out of more than 80 specimens, represented by this figure, be discarded, the curves for the three ages in the upper group would be in even more perfect agreement than the three curves in the lower group. This subject was touched upon in the discussion of the general phenomena of tests where the need for a large number of specimens for tests of this kind was pointed out.

In the light of the data in Fig. 15 it must be concluded that for the conditions of these tests, the hydration practically ceased upon removal of the specimens from the moist room. Such complete cessation of hydration upon removal of specimens from the moist room has not been found in the case of other forms of test as there is usually some increase in quality at least for a short time. An explanation in this case is probably found in the thickness of the specimens (1 in.), and in the humidity conditions of the laboratory air. The relative humidity in the room where these specimens were stored was usually between 50 and 60%, a condition which would be conducive to the rapid drying out of specimens only 1 in. thick. Some further tests are now under way in which the effect of air curing will be studied over a wide range of size of specimen and method of curing.

CHARACTERISTICS OF CEMENT

Studies of compressive strength and other properties of mortar and concrete have shown that all cements do not gain their strength at the same rate. Such differences as exist are more marked at the early ages than for the later period. In view of the importance of the early curing on the watertightness, which has just been brought out, it may be expected, therefore, that the permeability of concretes made from different cements would also differ at the early ages. In the data which have been pre-

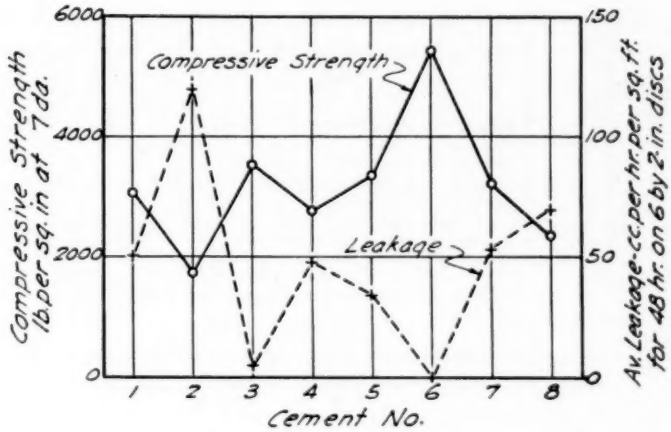


FIG. 16—STRENGTH AND PERMEABILITY OF CONCRETE MADE FROM DIFFERENT CEMENTS.

sented, separate curves for different cements have been given for several of the groups of tests. These have all shown some differences in leakage during the early period although the differences are not important in comparison with those due to the other factors.

This variation in early watertightness between concretes from different cements is illustrated further by the data in Fig. 16 where the results of tests at 7 days for 8 different cements are compared, both as to permeability and compressive strength. These 8 cements were selected to give concretes of a very wide range in early compressive strength. They include the customary laboratory cement, a mixture of 4 brands, several individual brands of portland cement and one of the special high early strength portland cements now on the market. The range in early strength for this group is very much wider than would be encountered ordinarily among portland cements. The purpose of this group of tests was to see how closely the development of watertightness parallels the development of compressive strength.

The permeability specimens on which tests were made were 6 by 2-in. discs of concrete; mix, 1:2.4:3.6 (dry compact volumes) using a water ratio of 6 gal. per sack. The specimens were cured moist for 3 days and then in air of the laboratory until tested at

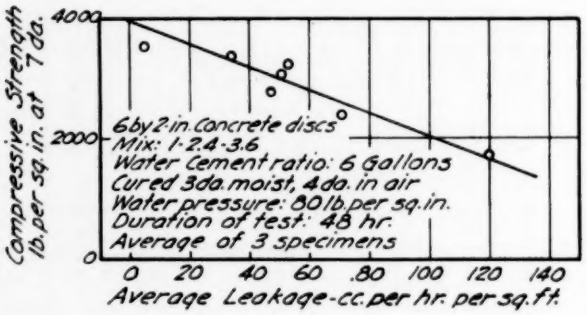

Compressive Strength lb. per sq. in. at 7 da.
4000
2000
0
6 by 2 in. Concrete discs
Mix: 1-2.4-3.6
Water Cement ratio: 6 Gallons
Cured 3 da. moist, 4 da. in air
Water pressure: 80 lb. per sq. in.
Duration of test: 48 hr.
Average of 3 specimens
0 20 40 60 80 100 120 140
Average Leakage-cc. per hr. per sq. ft.

FIG. 17—RELATION BETWEEN STRENGTH AND PERMEABILITY OF CONCRETE MADE FROM DIFFERENT CEMENTS.

the age of 7 days. Tests were continued for 2 days under a pressure of 80 lb. per sq. in. Companion compression specimens were given the same curing and also tested at the age of 7 days. Fig. 16 shows the results of these tests—compressive strength in lb. per sq. in. and the average leakage in cc. per hr. per sq. ft. for the 48 hr. under test.

It will be noted in Fig. 16 that in every case a cement which showed a high degree of watertightness at 7 days also showed a high 7-day compressive strength. In Fig. 17 these same data have been plotted, compressive strength against average leakage. This shows a very consistent relation between the 7-day strength of concrete and the leakage after 3 days moist and 4 days dry curing. The fact that these results are for the early ages must be kept in mind in this comparison. Other studies have shown that regardless of the differences between cements at the early ages, there is a general tendency for the strengths to approach each other at the later periods. In the two days under test, these cements all showed a rapid increase in watertightness, due in part to the improvement from continued curing. The same tendency was shown in Fig. 6 where the concrete made from the two different cements approached complete watertightness as the test was continued. Likewise the data in Fig. 7 and 9 also show the leakage in concrete from different cements to approach zero as the specimens continue under test.

While these data are somewhat limited they do indicate that the development of watertightness in the early period follows the

same general tendency as the development of early strength. In view of the rapid increase in watertightness with increased length of moist curing, the early differences in watertightness seem to be of less practical significance than differences in early strength.

THE EFFECT OF DIRECTION OF PRESSURE AND AMOUNT OF PASTE

In using the molded disc specimens the question arose as to the possible effect of molding in different positions. A small group of tests was made to study this factor. The results of these are shown in Table 1. For these tests two concrete mixes and two mortar mixes were used, 5 specimens of each being prepared by molding horizontally and 5 by molding vertically. The specimens were cured 3 days in the moist room and then removed to the air of the Laboratory where they remained until tested at the age of 7 days. Tests were made at a pressure of 80 lb. per sq. in. which was continued for a period of 2 days. The leakage shown in the table is the average leakage in cc. per hr. per sq. ft. for 48 hr. The individual results for the 5 specimens of a kind as well as the average of the 5 are shown.

TABLE I

	Vertically Molded Specimens				Horizontally Molded Specimens			
	Average Leakage cc. per hr. per sq. ft.				Average Leakage cc. per hr. per sq. ft.			
Gal. per Sack . . .	6	7	7	8	6	7	7	8
Mix	1:2:3	1:2½:4	1:3	1:3½	1:2:3	1:2½:4	1:3	1:3½
	—*	—*	299.0	1715.0†	90.5	169.0	79.4	397.0
	21.0	27.3	—†	149.2	14.5	131.8	120.0	278.0
	53.7	107.5	125.5	217.0	64.5	28.7	37.6	222.0
	17.2	79.0	21.4	630.0	6.1	43.0	301.0	724.0
	66.1	193.0	178.4	414.0	105.2	—†	105.0	378.0
Average	39.5	101.7	156.1	352.6	56.2	93.1	128.6	399.8

*Broke through pressure. †Hole through specimen.

In making the horizontally molded specimens the same procedure was followed as in the other tests of the series. The form consisted of a 6-in. galvanized iron ring 2 in. in depth. This was placed directly on an oiled metal slab, the concrete placed in the open end, and trowelled off. The specimens were covered for a period of 24 hr. with moist burlap. The forms were then removed

and the specimens placed in the moist room. Before test the top and bottom surfaces were ground off and brushed to a depth of about $\frac{1}{16}$ in. to remove the thin cement film which always forms in contact with the form and on the finished surface.

For the vertically molded specimens a similar form was used which was held rigidly in a vertical position between two oiled steel plates. An opening was cut in the metal ring through which the concrete or mortar was placed. This was placed in thin layers and puddled in the usual manner with a bullet-pointed rod. These specimens were cured in the same way and given the same treatment before test as those molded horizontally.

A study of the table shows very little difference in the average leakage between the vertically and horizontally molded specimens. These differences, it will be seen, are less than the differences between individual readings among the 5 specimens of each group.

Other tests have been reported in which greater differences between the two methods of molding have been noted. Obviously, such differences must arise from the method of distribution of the paste throughout the mass. In these tests, an effort was made to eliminate these variations by the selection of a consistency that would avoid air or honeycomb pockets on the one hand and water channels due to excess water on the other. By using mixes with a slight excess of sand and a consistency represented by a slump from 4 to 6 in., this desirable placing condition was obtained as is shown by the fair degree of uniformity already pointed out.

An interesting feature of this group of tests is the comparison between the 1:3 mortar mix and the 1:2¼:4 concrete mix, both using 7 gal. water per sack cement. It will be noted that the mortar shows consistently higher leakage for both methods of molding. The ratio of the leakage through the mortar to that through the concrete is 1.54 for the vertically molded specimens and 1.38 for the horizontally molded specimens. Calculations of the amount of paste in these two specimens for the 7-gal. water ratio shows that the mortar has 40.7% paste and the concrete 24.5%. This gives a ratio of paste in the mortar to that in the concrete of 1:66. For exactly the same quality of paste it would be expected, therefore, that the mortar specimens would

show a leakage of 66% greater than the concrete for the same total area exposed to test. This is somewhat greater than the 54% and 38% difference in leakage indicated above. The discrepancy, however, is not great and considering the fact which has been referred to several times that permeability tests are very sensitive to small differences, the data may be considered as quite consistent.

A further study of the effect of the quantity of paste on the leakage through concrete and mortar is afforded by the data in Table 2, which show a group of tests in which the water-cement ratio was held constant, but the quantity of paste varied. The specimens for these tests (6 by 2-in. discs) were given 3 days moist air curing before placing under the water pressure of 80 lb. per sq. in. While these tests were made primarily to study the effect of quantity of paste, certain other features were developed which are of interest in this connection. These are brought out from a study of the table.

TABLE II

EFFECT OF AMOUNT OF PASTE ON PERMEABILITY OF CONCRETE

Mix by Corrected Volumes	Water-Cement Ratio Gal. per Sack	Slump In.	Percent Paste by Vol.	Average Leakage for 2 Days cc. per Sq. Ft. of					
				Concrete					Paste
				A	B	C	D	Av.	Av.
1:3½	8.0	4–6	39.1	460	1200	404	750	704	1800
1:3:2	8.0	4–6	31.5	270	272	257	166	241	765
1:2½:4	8.0	4–6	26.3	232	221	99	136	172	655
1:2½:6*	8.0	0	21.5	44	123	105	50	81	375

*Harsh and stiff, jigged into molds.

The mixes used show a range in paste volume from 21.5% to 39.1%. To get this range with a constant water-cement ratio required at one extreme, a very harsh stiff mix which could not be placed except by the aid of mechanical means, and at the other extreme, a mortar mix which segregated badly allowing water to collect at the surface. These extremes introduced the element of placing in the test. This shows up in the higher leakages for the first mix and the lower leakages for the last mix in the table,

also in the greater variation between individual specimens for these two mixes.

Referring to the effect of quantity of paste, this is brought out by the averages in the last two columns. When expressed in terms of the gross area of the concrete, the leakage per unit area falls off rapidly with the decrease in the proportion of paste. When expressed in terms of the net paste area, however, it is seen that the leakage is more nearly the same for the four mixes. The discrepancies in the case of the first and last mixes are, of course, quite important but seem to be accounted for by conditions that developed in placing.

EFFECT OF ADMIXTURES ON PERMEABILITY

The effect of a number of different kinds of admixtures on the permeability of concrete has been studied in two different series of tests. These studies have not been completed, but numerous data have been collected which will be of interest here.

The first study of the admixtures included three different concrete mixes—1:1½:3, 1:2:4, and 1:2½:5—each containing two percentages of admixtures, but having about equal consistency as measured by the slump cone. The slump was maintained at 3 to 6 in. throughout the series. The admixtures used in this series were purchased in the open market and are identified as follows:

Admixture No. 1 purchased 7-27-25 as Celite
Admixture No. 2 purchased 3-29-27 as Pumicite
Admixture No. 3 purchased 8-10-27 as Hydrated Lime
Admixture No. 4 purchased 7-5-28 as Soapstone

The permeability specimens were 6 by 2-in. discs containing aggregate of 1½-in. maximum size. They were cured 7 days in the moist room and then removed to the air of the laboratory until tested at the age of 28 days. The specimens were tested under a water pressure of 80 lb. per sq. in. for 2 days, after the surfaces had been ground and brushed to remove the coat of cement. The results obtained are plotted in Fig. 18, the ordinates representing the average leakage for 48 hr. in cc. per hr. per sq. ft., and the abscissas representing, to no scale, the three mixes used.

The percentage of admixture used was selected to exceed slightly the range ordinarily used. In the case where portland

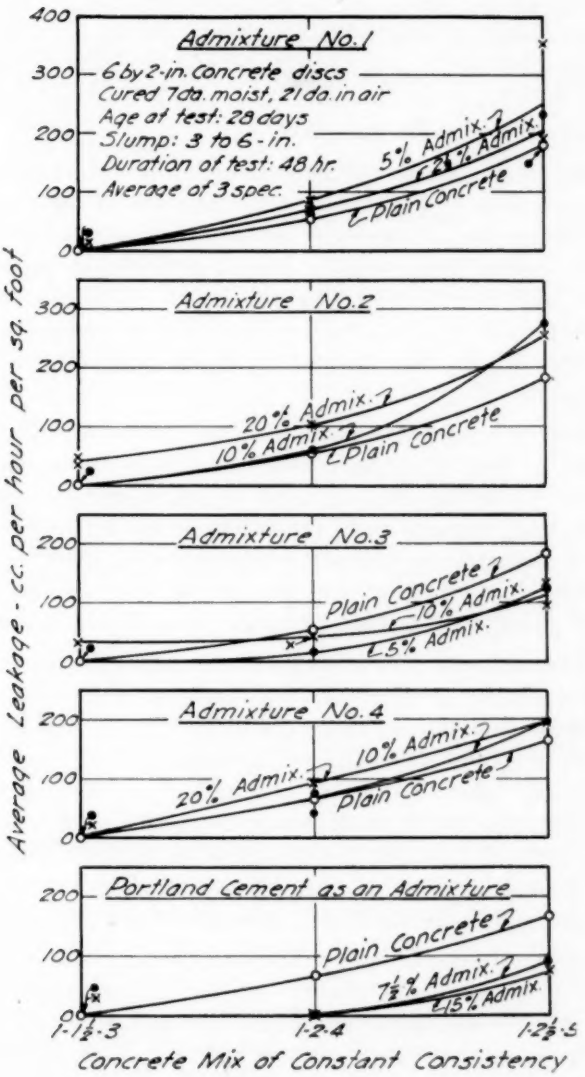


FIG. 18—EFFECT OF ADMIXTURES ON PERMEABILITY OF CONCRETE OF CONSTANT SLUMP.

cement was used as an admixture, the percentages were chosen to give about the same cost as for admixture No. 1. Expressed as a percentage by weight of the cement, the amounts of admixtures were 2½ and 5% for No. 1, 10 and 20% for No. 2, 5 and 10% for No. 3, 10 and 20% for No. 4, and 7½ and 15% for portland cement used as an admixture.

In order to maintain constant slumps when adding admixtures, it was necessary in most cases to increase the amount of mixing water. While the extra water added for No. 3 and 4, and for portland cement was slight, No. 2 required a considerable increase, and No. 1 a still greater increase in the water-cement ratio over that used in the concrete without admixture. An addition of 5% of No. 1 raised the water-cement ratio about 1 gal. per sack.

The principal results of these tests are shown in the 5 separate diagrams of Fig. 18, representing the results from admixtures No. 1 to 4 and portland cement as an admixture. On each diagram, the leakage curve for concrete without admixture is repeated. This makes it easy to compare the results of the various admixtures. In studying this diagram it must be kept in mind that the mixes were of constant consistency (slump 3 to 6 in.).

The two features of importance in this set of diagrams are the general similarity of all the leakage curves and the relative leakage of the concretes with and without admixture. In practically all cases, the 1:1½:3 mixtures show substantially no leakage, while for the other two mixtures there is a considerable increase which is greater for the leaner mixture. This, of course, is just another way of bringing out the relation between leakage and quantity of mixing water, for the leaner mixes require higher water contents to maintain constant consistency.

It will be noted that in the diagrams for admixtures 1, 2, and 4, the leakage curves lie slightly above that for plain concrete, while in the remaining two diagrams the concrete specimens with no admixture show slightly greater leakage. It was pointed out above that in order to maintain a constant consistency the concrete containing No. 1 and 2 required considerably more water than the concrete without admixture, while that for No. 3 and 4 required practically no additions. The slightly greater leakage

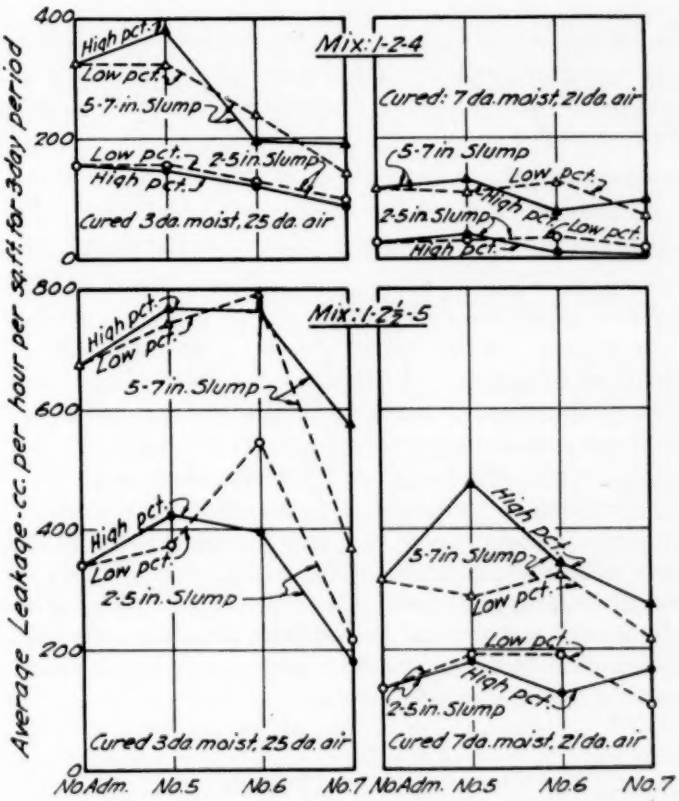


FIG. 19—EFFECT OF ADMIXTURES ON PERMEABILITY OF CONCRETE.
FIG. 20—RESULTS OF FREEZING AND THAWING TESTS.

shown in the two upper diagrams of Fig. 18 seems, therefore, to be the direct result of extra water added to maintain the same consistency as in the concrete without admixture. In the diagram for admixture No. 4, the very slight amount of extra water needed is reflected in the very slight increase in leakage. In the case of No. 3, there seems to be a slight improvement in watertightness. For those mixes where small percentages of portland cement are added as an admixture there is a distinct improvement. In the case of the portland cement additions. very little extra water was required to maintain workability.

In Fig. 19 are given the data from the second series of tests, in which two mixes—1:2:4 and 1:2½:5—were used each in two consistencies and with two different moist curing periods. The admixtures used in this series were purchased in the open market and are identified as follows:

Admixture No. 5 purchased 1-24-29 as Celite
Admixture No. 6 purchased 1-24-29 as Pumicite
Admixture No. 7 purchased 1-24-29 as Hydrated Lime

The results in Fig. 19 are plotted in a somewhat different form which brings out even more clearly than Fig. 18 the effect of the admixtures. In this figure, the two upper diagrams are for the 1:2:4 mixes and the two lower ones for the 1:2½:5 mixes. In each pair, the diagram at the left represents specimens cured 3 days in the moist room and 25 days in ordinary air, and the diagram at the right, 7 days in the moist room and 21 days in air. In this figure the points representing identical conditions for the different admixtures have been connected so that for any given condition the difference in leakage for the different admixtures can be easily studied. Thus, in each diagram there are curves to represent the two percentages of admixtures and two consistencies. The high and low percentages in each case are in accordance with the following table:

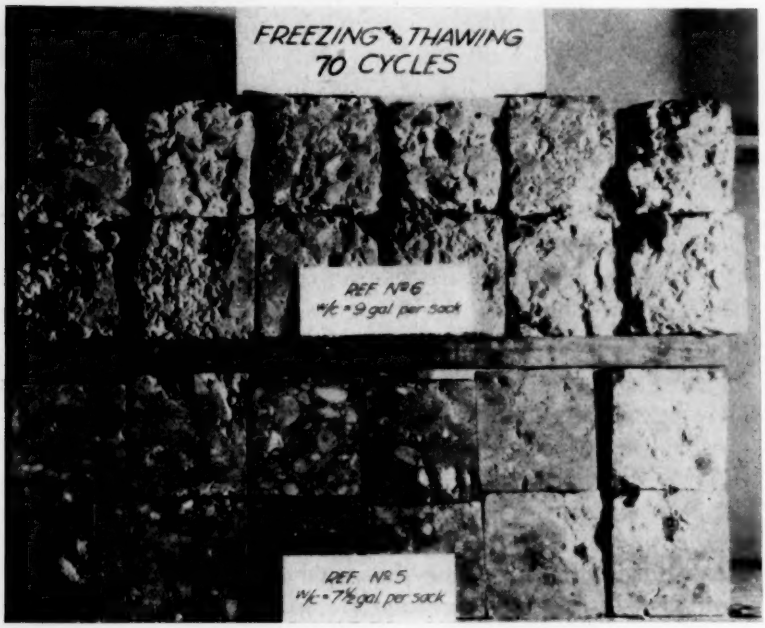
Percentage by Weight	Admixture		
	No. 5	No. 6	No. 7
Low	2½	10	5
High	5	20	10

At the extreme left of each diagram the leakage for the identical mix without admixture is shown. Naturally, the curves for both high and low percentages start at this same point. The comparison between the concretes with and without admixtures can be seen rather clearly in these diagrams. Just as pointed out in the discussion of Fig. 18 for admixture No. 1, there is some slight increase in leakage for the use of No. 5 which seems to reflect the effect of additional water required to maintain constant consistency. The same is true of No. 6, except in the case of one

or two points which indicate a slight advantage in watertightness. In the case of No. 7 a slight advantage is shown as was also found for No. 3 in the tests of Fig. 18. It is important to note that the irregularity in the position of the curves for the two percentages of admixture is an indication of the sensitive character of the permeability test. The difference between individual specimens of a kind is sometimes greater than the difference indicated on the diagram for the different admixtures or percentages. It is of interest also that both the irregularity for a given set of curves and the differences shown by the different admixtures are reduced by the longer curing.

In studying these data, the slight difference in leakage between the concretes containing various admixtures, and without admixtures, must not be given too great emphasis. In comparison with the differences resulting from changes in the much more significant factors of consistency and curing, or from improper placing, they are quite unimpressive. For example, the improvement resulting from a change in water content that reduces the slump from 6 to 3 in. is somewhat greater than that possible from the most favorable combination of admixture, and very much greater than from the other combinations. In addition, such a change in water content will be accompanied by a decided increase in strength. Likewise, the increase in watertightness due to increasing the moist curing period from 3 to 7 days is considerably greater than for the most favorable combination of admixture and in this case also the improvement would be accompanied by a positive improvement in strength.

It would seem from the foregoing that if admixtures are to be particularly useful in the production of watertightness in concrete, it must be through their effect on the placeability. As has been pointed out elsewhere, the greatest source of leakage in concrete structures is through defects in placing. These defects are generally of two kinds, those due either to overwet mixes or to mixes that are too dry. In the former, there is a wet segregation, water, fine material, and laitance rise to the surface producing an extremely porous layer. In the latter, there is a dry segregation and tendency to honeycomb which may leave open channels for the passage of water. These conditions can be



overcome only by selection of proper aggregate proportions and consistencies and care in placing. The concrete must be such that it will readily fill the form and incorporate the reinforcement, and at the same time maintain such plasticity and cohesiveness that segregation will not take place. It is recognized that under certain conditions some of the admixtures may be helpful in producing this desirable consistency and to this extent aid in the production of watertight concrete through elimination of placing defects. In their use, of course, the other necessary qualities must be maintained and such use must be justifiable economically.

FREEZING AND THAWING TESTS

Parallel with the permeability tests, studies have been carried out to determine the factors affecting the resistance of concrete to freezing. The results of one group of these tests are included here which seem to be of special interest as they confirm and illuminate some of the permeability data. It is recognized that the freezing and thawing as carried out in these tests is, undoubt-

edly, a much more severe condition than the exposure usually encountered. It is a fair basis, however, for comparisons similar to that made here.

Fig. 20 shows the photograph of 24 6-in. cubes taken after exposure to 70 cycles of freezing and thawing. The cubes were cut from beams of 1:2:4 concrete made up in two water ratios, 7½ and 9 gal. per sack. The lower water ratio gave a slump from 6 to 7 in. while 9 gal. per sack gave such extremely wet mixes that no effort was made to measure the slump. The specimens were cured in the moist room at 70° F. for 28 days before being subjected to the freezing and thawing tests. During the entire period of freezing and thawing, the specimens were kept in a saturated condition. It will be seen that some of the cubes of the wetter mixture are badly disintegrated, while those of lower water ratio are still in excellent condition. The few defects which have developed in the specimens of this better consistency are almost entirely at the corners and edges. The faces of these cubes show very few defects at the end of this severe treatment, and those that do show are principally due to aggregate failure.

Half the specimens of each group shown in Fig. 20 were tested in compression after the picture was taken. The age at test was 13 mo., the 70 cycles having required about 12 mo. to complete. The results of the compression tests are shown in the following table.

Specimen	Water Ratio Gal.	Weight of Specimen	Compressive Strength Lb. per Sq. In. Gross Area
5–1	7½	20.3	5700
2	7½	18.2	6100
3	7½	20.2	4960
4	7½	19.7	5250
5	7½	19.6	5200
6	7½	18.2	4950
		Av. 19.4	5360
6–1	9	17.6	2680
2	9	16.7	1490
3	9	15.0	1190
4	9	14.9	960
5	9	19.7	3080
6	9	18.5	2520
		Av. 17.1	1990

The data are of particular interest when compared with the strengths shown at 28 days by companion specimens in the form of standard 6 by 12-in. cylinders. The cylinder tests showed a compressive strength of 2420 lb. per sq. in. for the 7½-gal. mix, and 1280 lb. for the 9-gal. mix which is equivalent to 2840 and 1500 lb. per sq. in., respectively, on the basis of cube specimens. Comparison of these values with those in the table, shows that the 12-mo. freezing and thawing tests have increased the strength of the 7½-gal. mix by 90% which is about the normal increase in strength due to moist curing at 70° F. It is interesting to note also that there is a fair degree of uniformity between the 6 specimens of this group, showing that the effect of the 70 cycles has not begun to show on any of the specimens.

In the case of the 9-gal. mix, the great variation between the compressive strengths of the 6 specimens will be noted. This is to be expected from an examination of the picture which shows that some of the specimens have been more seriously affected than others. In this case, the effect of additional curing, however, is still very pronounced, for only 3 specimens of the group actually fall below the original 28-day strength (on the basis of cubes) of 1500 lb. per sq. in. It is to be pointed out that the strengths given in the table are based on the gross area of 36 sq. in. of the cube. If it were possible to estimate accurately the net area at the section of failure the specimens would, undoubtedly, show unit strengths much greater than the original value and probably a greater uniformity.

SUMMARY

This investigation has developed a method of making permeability tests that is quickly and easily made and which gives results in terms of water actually passing through the concrete. The data obtained have been very consistent and give every indication of being reliable.

The studies have shown that the permeability test is very sensitive. Minor defects in concrete that would make no appreciable change in compressive strength affect the flow of water through the specimen under pressure to a marked degree.

The tests have demonstrated what has been observed in concrete structures that defects in placing are a much more

important factor in the leakage than ordinary differences in mixtures, materials, or water content.

The most significant results of the tests are those showing the effect of continued moist curing in increasing the watertightness of concrete. It was early demonstrated that for all the ordinary mixes encountered in practice the usual 28-day moist curing at 70° F. produced concrete through which no water would flow under pressures up to 120 lb. per sq. in. It was, therefore, necessary to use much shorter curing periods to obtain test results that would permit comparisons between the other variables under consideration.

Tests using these shorter moist curing periods showed very rapid increase in watertightness with increase in length of moist curing period. These increases held, regardless of the other factors of the tests, showing that the length of moist curing is a primary factor in the building up of watertightness of concrete.

This investigation has clearly shown the important part played by the quantity of mixing water in watertightness. This is second in importance only to the length of moist curing. An increase in the quantity of mixing water was accompanied by a reduction in watertightness, regardless of the curing period or the characteristics of the cement used.

The two factors of water-cement ratio and period of moist curing are so intimately related in the building up of the structure of the hardened cement paste that it is impossible to give the limits of one without stating also the limits of the other. For the equivalent of 3 days moist curing at 70° F., concrete and mortar with a water-cement ratio of 5 gal. per sack or less showed no leakage of water under pressures up to 80 lb. per sq. in. For an equivalent of 7 days moist curing at 70° F. the water ratio necessary to produce such complete watertightness was found to be 6 gal. per sack. Water-cement ratios of 9 gal. per sack produced complete watertightness for 28 days curing. Water-cement ratios greater than 9 gal. were not used in these tests. Further tests will be necessary to establish the permissible limits under this extended curing period.

The tests have also been very illuminative in showing that after specimens were removed from the saturated atmosphere of

the moist room and placed in an atmosphere of about 50% relative humidity, there was practically no increase in watertightness. This was true of specimens tested at various periods up to 6 mo. In interpreting this in connection with the usual concrete practice, it must be remembered that these specimens were thin discs which are readily affected by external conditions. It is not likely that conditions so extreme would ordinarily be encountered in concrete practice where the more massive sections prevent the rapid loss of moisture. The tests are significant, however, in showing that progressive building up of the internal structure of the concrete through curing cannot take place unless moisture is present for the continued chemical reactions.

The tests using cements of different characteristics showed that the development of watertightness at the early ages was comparable with the development of compressive strength. Those cements which gained their strength rapidly during the first few days were found also to gain watertightness more rapidly. With continued moist curing all the specimens of moderate water-cement ratio became completely watertight within a few days.

Studies of some of the common powdered admixtures showed that additions of those materials which required extra water to maintain plasticity reduced the watertightness, while with those materials which required no extra water there was some slight improvement. The changes brought about by these additions, however, are quite insignificant in comparison with the effects produced by a few days change in the period of moist curing or a moderate reduction in the water content.

The results of the freezing and thawing tests are very illuminative in showing the marked advantage of a low water-cement ratio in increasing the resistance of concrete to freezing.

Readers are referred to the JOURNAL *for April, 1930, for discussion which may develop. Such discussion should reach the Secretary by March 1, 1930.*

Concrete Roadbed On The Pere Marquette Railway

BY PAUL CHIPMAN*

Increasing traffic and heavier wheel loads have for some years past directed attention to the insufficiency of the usual type of railroad track. The prime defect of a rail support consisting of ties and ballast is its lack of continuity. The assumption has been general, however, that a rigid and continuous track support is not practicable. Street railways were built in which the rail rested on and was embedded in concrete, thus providing a perfectly rigid track; but it was taken for granted some yielding in the track was a necessity under the heavy loads and high speeds of steam railway traffic. For this reason also, even such installations of concrete roadbed as were made, either experimentally or in special locations such as station grounds and tunnels, provided for carrying the rail on wooden blocks set in the concrete base.

Several years ago Mr. Frank H. Alfred, until recently president and general manager of the Pere Marquette Railway Company, became convinced from his study of the subject that a rigid concrete roadbed was entirely practicable and would result in substantial economies and that these economies would be sufficient for railroads carrying very heavy traffic. to justify the admittedly high cost of installation.[1]

THE FIRST INSTALLATION

With a view to verifying its practicability by actual experiment, the Pere Marquette in 1926 installed a section of concrete roadbed at Beech, Mich., about twelve miles from downtown Detroit, on its westbound main track between Detroit and Grand Rapids. It is 1326 ft. long, being cast in 34, 39-ft. sections. The construction is shown in Fig. 1. The cross section is essentially a rect-

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[1]Permanent Railroad Track, by Frank N. Alfred, *Proceedings American Concrete Institute*, Vol. 22, 1926, page 45.

angle 21 in. deep and 10 ft. wide. The ends of adjacent slabs are connected by a horizontal tongue and groove joint, the object of which is to prevent independent vertical movement. The reinforcement consists of a light truss under each rail, in addition to ordinary reinforcing bars placed longitudinally and transversely. The upper chord of the truss consists of two ¼-in. by 4-in. steel plates, separated by 3-in. channels which constitute the vertical and inclined members of the truss. The bottom chord consists of two ¼-in. x 1½-in. angles. The upper edges of the ¼-in. x 4-in. metal plates were intended to provide a metal rail seat, as it was suspected that unless a metal seat were provided, the concrete would be abraded and perhaps disintegrated by the motion of the rail and the resulting impact.

When construction began, however, it was found that the distortion of the steel work incidental to its fabrication was such as to make it unsuitable as a rail seat and that to obtain a true plane it was necessary to finish the concrete a little higher than the edges of the metal plates. The rail would thus rest directly on the concrete, except for a few spots where the steel came to the surface. This circumstance was most fortunate, as it led to the discovery that the rail could rest directly on the concrete without producing the undesirable effects that had been anticipated. After nearly three years of service there has been no appreciable disintegration or wearing down of the concrete surface upon which the rail rests.

The steel trusses referred to are connected by cross braces and tie rods at intervals of about 12 ft. and additional tie rods are placed intermediate of these cross braces. At proper intervals stirrups made of ½ in. x 2 in. steel are bolted to the upper chord of the truss to provide means for attaching the rail fastenings. These fastenings consist of ordinary rail-clips and bolts. A nut is embedded in the concrete beneath the flange of the stirrup and the bolt with head up is passed through the rail clip and the stirrup and screwed into the nut.

The concrete was proportioned on the water-cement ratio basis to produce about 4500-pound concrete. An average of about 4.2 gallons of water per sack of cement was used, and about 9 sacks of cement per cubic yard. The average slump was about

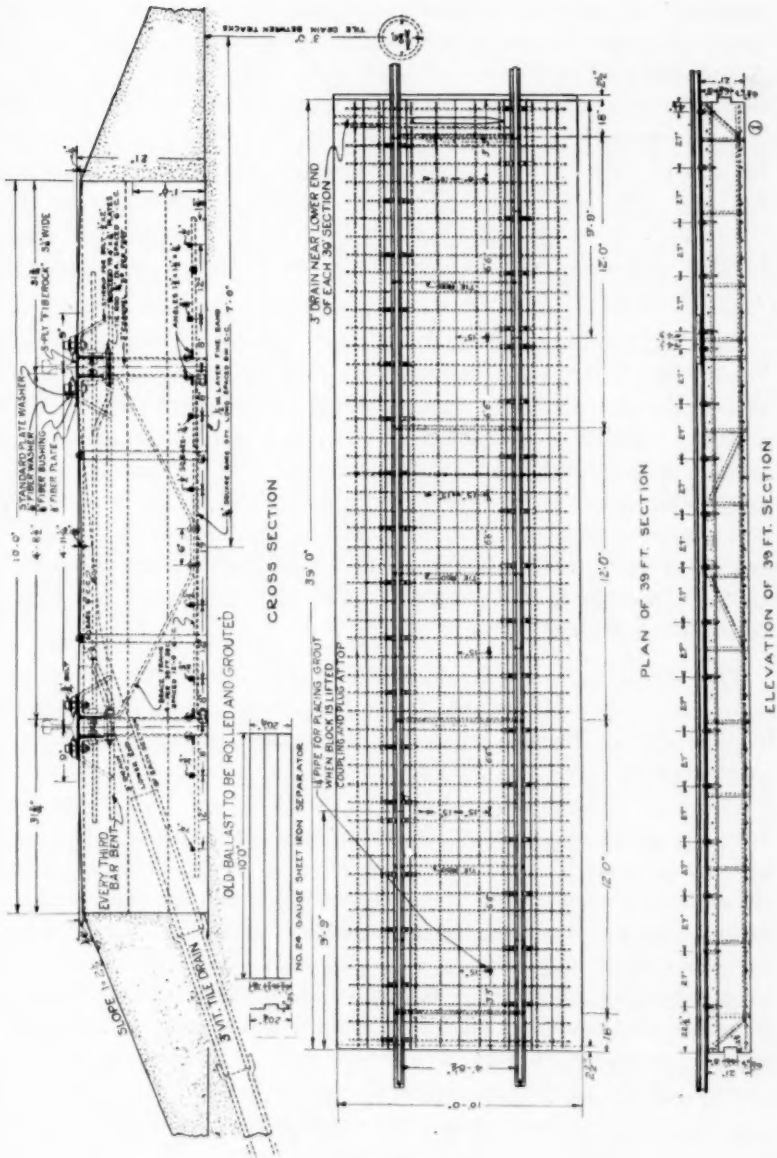
CROSS SECTION
PLAN OF 39 FT. SECTION
ELEVATION OF 39 FT. SECTION
OLD BALLAST TO BE ROLLED AND GROUTED
EVERY THIRD BAR BENT
SLOPE
3" VIT. TILE DRAIN
NO. 24 GAUGE SHEET IRON SEPARATOR
3" DRAIN NEAR LOWER END OF EACH 39' SECTION
39' 0"
12' 0"
10' 0"

FIG. 1

4 inches. The mixing, proportioning and laying of concrete was under the supervision of a representative of the Portland Cement Association. Three test cylinders were taken from each 39-ft. slab. It was intended to test one cylinder from each slab at 7 days, one at 28 days and one at 3 months, and this was done for the 28 day and 3 month specimens, which averaged 3605 lbs. and 5083 lbs. respectively. It was not possible to test all of the specimens intended for the 7-day test at that age. The average strength of these specimens was 2719 lbs., obtained at an average age of 10 days.

The aggregate consisted of washed gravel and sand from Green Oak, Mich.—the gravel from ¼ in. to 1½ in; sand coarse and fairly sharp. The placing of concrete began October 25 and was completed November 27. In this period the temperature ranged from 22 degrees F. to 65 degrees F. When the temperature approached the freezing point the water was heated, and when it fell to 25 degrees or lower the sand was also heated by piling it around a stationary boiler tube in which fire was maintained. Curing was done by covering with sand and loam, kept moist by sprinkling when necessary.

The laying of rail was completed a few days after the last concrete was placed and on December 19 the track was placed in operation. Levels taken before operation began and on December 30, when the track had been in service 11 days, showed a settlement ranging from ⅛ in. to ¾ in. However, this difference in settlement does not occur between slabs which are adjacent or even near to each other. It appears to be due to conditions in the subgrade which extend for a considerable distance and which do not change abruptly, as the greatest variation between the two ends of the same slab was ⅜ in. and this only in two slabs. In general this difference does not exceed a ¼ in. These differences are not visible to the eye and cannot be detected in riding over the track on a train.

When plans were drawn and the material ordered a different location was in mind than the one finally chosen. The latter is in block signal territory and it was therefore necessary to provide some way of insulating the rails from each other. The arrangement of a metal rail support with connecting metal braces and

tie rods was ideal for conducting current between the opposite rails. Insulation was provided by placing a layer of ⅛-in. pressed wood insulating fibre under the north rail in addition to providing fibre bushings and washers for the bolts.

While the ⅛-in. layer of fibre was installed under the north rail for the purpose of insulation, it has been found to have other advantages. There has been no tendency for the north rail to creep, but with the south rail there has been some trouble in this respect. The insertion of a layer of softer material between the rail and the concrete affords a grip for the bolts and keeps them tight, whereas if the rail rests directly on the concrete a slight stretching of the bolts due to vertical motion of the rail causes them to lose their holding power.

In the spring following construction, transverse cracks appeared in a few of the slabs. In only four of them do these cracks extend for the full width of the slab. One of these cracks is at a construction joint in the first slab, which was not completed the first day. Another crack is in the eleventh slab from the west end, for which no cause is apparent, unless it be that two different brands of cement were used, one for each half of the slab. The other cracks are in the two slabs next to the east end. These were placed when the weather was quite cold and the temperature was low for a good part of the time until they were put in service, so that they had probably not attained the strength of the other slabs when operation began. None of these cracks has widened and after three years no additional cracks have formed.

Three years of operation over this track seem to disprove the contention that the impact of passing trains will shatter a concrete rail support if in direct contact with the rail. Perhaps a trial of this duration is not entirely conclusive and it is barely possible that effects of this kind will develop later; but the concrete under the rail does not show any indication of them. It would seem if deterioration were going on that it would be apparent by this time, but such is not the case. Batter of the rail at the joints has not been excessive and averages about the same as on ordinary track. The track rides very smoothly.

The worst objection from the passenger's point of view is the slight feeling of rigidity that is experienced, together with the sound caused by the reflection of the noise of the wheel grinding on the rail and the rattle of equipment. The flat surface of the slab seems to act as a sounding board, with an effect somewhat similar to that noticed in passing a freight train.

THE NEW INSTALLATION

After more than two years' observation of this roadbed some rather definite ideas were formed as to how it might be improved and at the same time reduced in cost. In the first installation the principal consideration was to build with sufficient strength to meet all possible contingencies and economy of design was of secondary importance. In the old design the unit compression in the concrete corresponding to a tensile stress of 16,000 pounds in the steel is only about 350 pounds, giving a factor of safety of about 12. The rectangular cross section is not economical of concrete. It had also been found that a metal rail seat is not necessary. Our experience indicated that a layer of slightly compressible material between the rail and the concrete is desirable, not as a protection for the concrete, but to afford a better grip for the rail fastenings and more comfort for the passenger by doing away with the sensation of rigidity that is experienced if the rail rests directly on the concrete. Doing away with the metal rail seat made it possible to eliminate the use of structural steel and to use only ordinary reinforcing bars. Plans were therefore drawn for a simpler and more economical design, which may be described as a platform resting on the subgrade, from which arise two longitudinal girders, one under each rail. These girders are tied together at intervals of about 6 ft. by means of cross-walls reinforced with loops of steel. See Figs. 2 and 3.

In the first installation expansion joints were placed at intervals of 39 ft., this being the usual length of a rail, the rail joint coming at the quarter points of the slabs. In the new design the sections were made 19 ft. 6 in. long, with the rail joint at the center of alternate slabs. The former design having proven sufficiently strong, it was used as a basis for computing the reinforcement in the new design. The depth from base of rail to bottom of slab is 20 inches, whereas in the old design it is 21

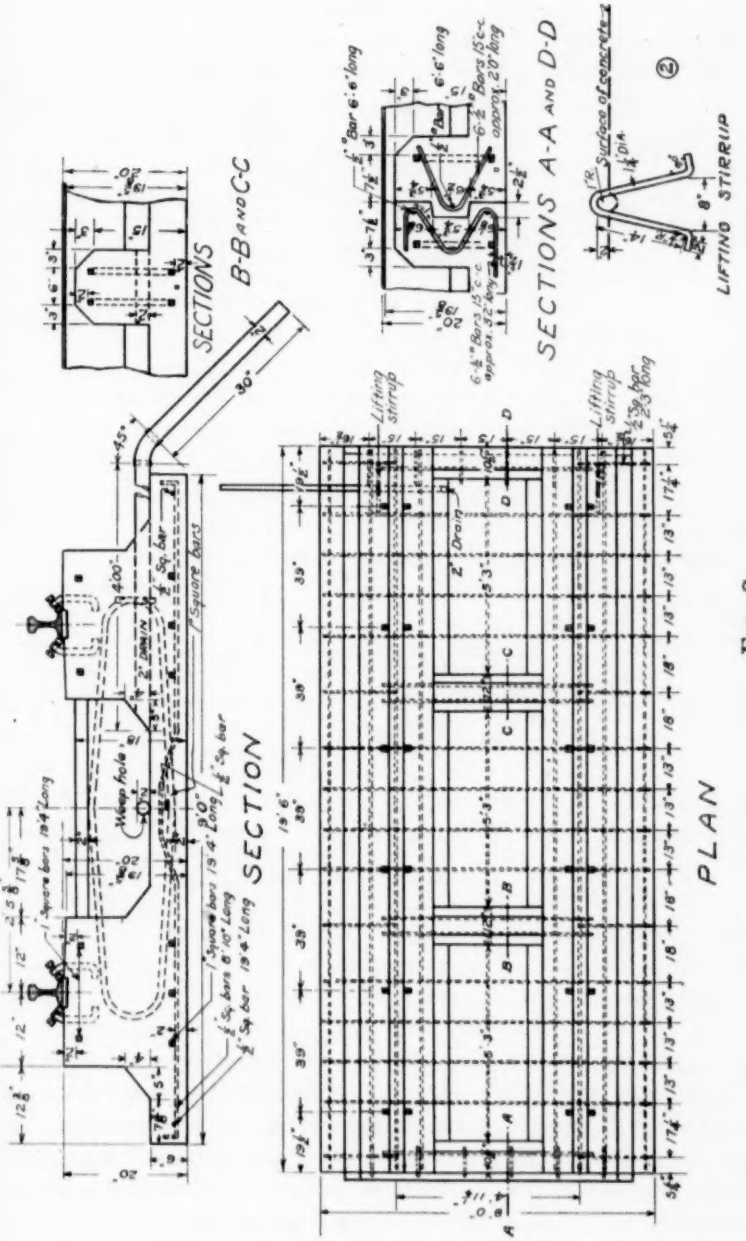
SECTIONS B-B AND C-C
SECTION
PLAN
SECTIONS A-A AND D-D
LIFTING STIRRUP
Weep hole
Square bars
Lifting stirrup
Drain
Surface of concrete

FIG. 2

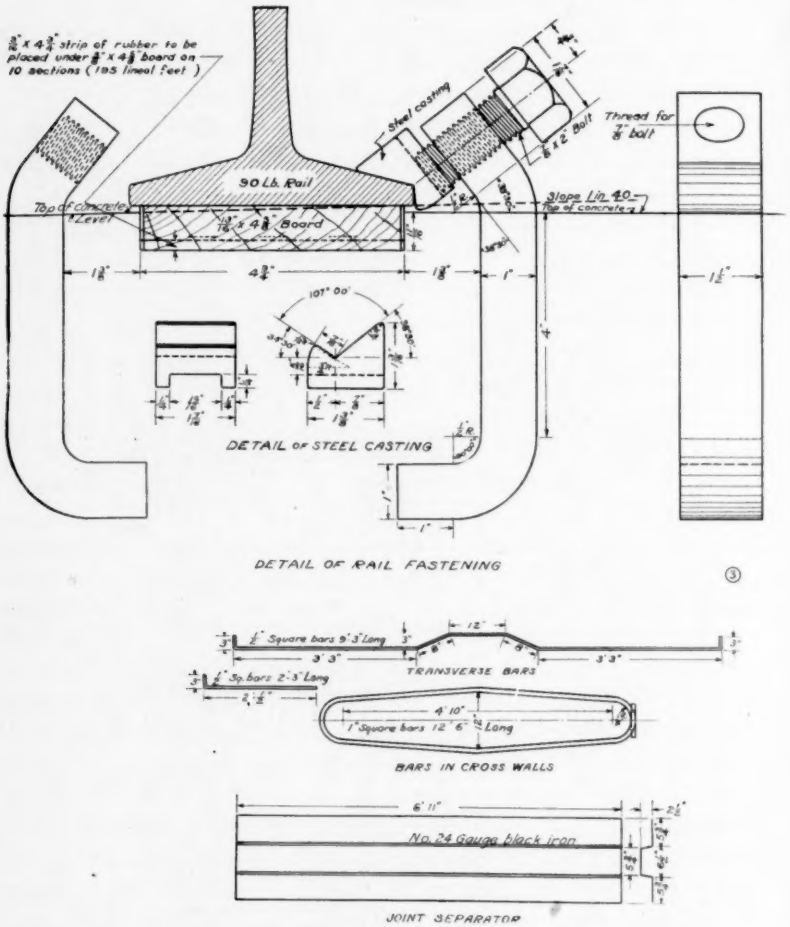

FIG. 3

inches. The width of the footing is 9 ft. instead of 10 ft. as in the old one. The new design calls for about the same amount of steel per linear foot of track as did the old one, but owing to the fact that ordinary bars are used, considerable economy is effected. There is a reduction of about 40% in the amount of concrete required per foot of track, which now amounts to ⅜ cu. yd.

The horizontal tongue and groove expansion joints, having proven satisfactory, were retained in the new design. They were made by using a separator of No. 24 gauge sheet iron bent to the required form and backed up by ¼-in. Elastite. Under each rail a shallow depression is provided, slightly narrower than the rail. In this depression a creosoted board is placed slightly thicker than the depth of the depression, with the rail seated on this board. This provides a grip for the rail fastenings and a slight cushioning effect which has been found desirable.

At intervals of 39 inches anchors made of 1-in. x1½-in. steel are embedded in the concrete, the portion which extends above the concrete being inclined toward the rail. A short bolt is threaded through this inclined portion and its end bears against a small steel casting which rests on the shoulder of the rail base. By tightening the bolt on one side and loosening the one on the other, a lateral adjustment is provided. Such adjustment is necessary, as it is impossible to set the anchors exactly in line. By tightening both bolts a downward pressure is exerted, which holds the rail firmly and prevents excessive vertical motion.

The space between the girders is filled with gravel and a shoulder of sand and gravel is built up for a width of about 18 inches beyond the outer edge of the girder, to lessen the danger in case of derailment, to afford better protection from frost and to do away with the reflection of noise.

The new installation is 390 ft. long, consisting of 20 19-ft. 6-in. sections. The work of placing occupied about three weeks and was completed September 4, 1929. Operation over it began September 20. It is just west of and adjoining the old installation.

Traffic was removed from the track to be improved by diverting it to the adjacent main. The traffic on that main was in turn diverted to an adjacent passing track, thus continuing double track operation. The subgrade was prepared by removing the track and shoveling off the ballast to the proper elevation, which in general was about the level of the base of the old ties.

The work was done by contract on a percentage basis. The aggregate was unloaded from cars set near the middle of the work before the old track was taken up. Water for mixing and curing the concrete was obtained from a locomotive tank

placed on a temporary siding at the west end of the work, where the cars of cement were also placed.

In the new section the same kind of aggregate was used and from the same source as before. Concrete was designed for a strength of 4000 pounds per square inch at 28 days, using the water-cement ratio method. An average of about 4¾ gallons of water was used per sack of cement and about 8 sacks of cement per cubic yard. The work of mixing and placing was again under the supervision of the Portland Cement Association. Two test cylinders taken from each slab were tested at 40 days and averaged 4650 pounds per square inch, the highest being 5580 and the lowest 3460. Curing was in damp sand until shipped to the laboratory where it was continued in a moist room until 48 hours before testing. The low strength of the weakest cylinders is explained by the fact that in each case a large piece of friable material was found at the apex of the cone break.

Sectional forms of wood were built at the Pere Marquette shops in Grand Rapids prior to beginning construction. Five sets of forms were made and after using them for the 20 slabs, they were still in fair condition. Two by four stakes were driven at the sides of the roadbed and the forms supported by cross pieces spiked to these stakes. Inasmuch as the surface of the rail, and to some extent its line, is governed by the accuracy with which the depression in the top of the girders is formed, much care was taken in this respect, both as to line and surface and as to cant, or transverse inclination. Furthermore, it was necessary to suspend the anchors in correct position and hold them firmly while the concrete was being placed. To accomplish these ends a sized 4¾ x 5 in. timber was used, being set to line and grade by transit and level and spiked to the cross pieces. A satisfactory surface was obtained for the bottom of the groove, but air bubbles were not entirely eliminated.

Experience with the original installation, where there has been no appreciable inequality of settlement, indicates that adjustment of surface would not be necessary except at long intervals and probably in a limited number of unfavorable locations. However, there would undoubtedly be certain places where settlement would necessitate occasional lifting of the slabs in order to restore

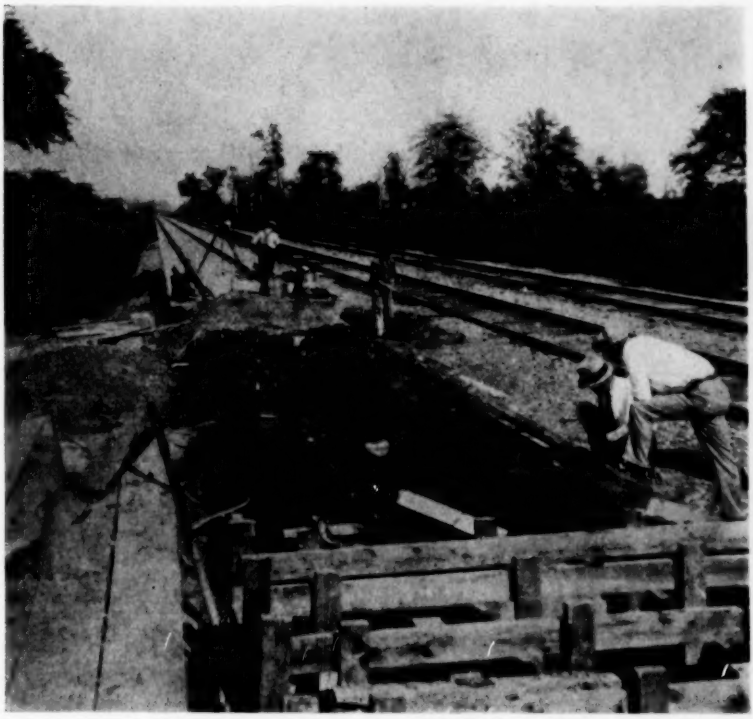

FIG. 4. CONCRETE ROADBED UNDER CONSTRUCTION

them to their former elevation and to remove slight inequalities of surface in the track. To facilitate this, two lifting stirrups are set in the concrete near one end of each slab. These are located between the rail and the outer edge of the girder, one on each side. They are made of 1¼-in. round rust-proof iron. Should it become necessary to lift a joint, an I-beam would be placed across the track and hooked to these stirrups. Jacks would be placed under the ends of this I-beam and the joint raised to the desired elevation. Sand would then be forced into the space between the sub-grade and bottom of the slab by compressed air. For this purpose the cement gun is a very satisfactory implement; as demonstrated by experiments made in using a wooden platform

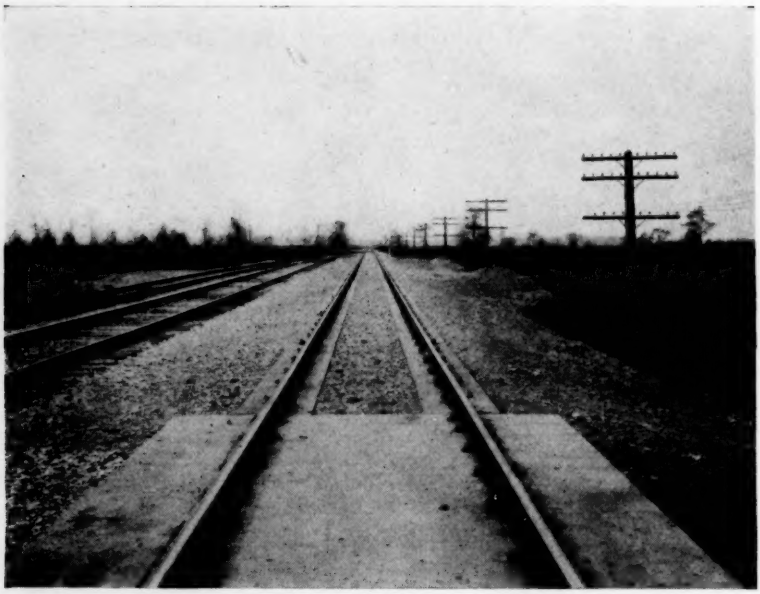

FIG. 5. COMPLETED—OLD AND NEW SECTIONS

in lieu of the concrete slab. The sand packs solidly and uniformly. For doing this work on a large scale, the equipment would consist of a motor car carrying the I-beam referred to, with jacks attached to it and operated by the engine of the motor car. Following this would come a flat car carrying an air compressor, two cement guns and a supply of sand. If traffic could not conveniently be diverted, it would be necessary to have a locomotive with the car. With such an outfit the work of re-surfacing could be very quickly and economically done.

The estimated cost of the new design is about $40,000 per mile, not including rail. Ninety pound rail and fastenings would bring the total to $48,000. This estimate is based on the cost of doing highway work by modern methods and with modern equipment, including steel forms. The construction requirements are very similar.

Improvement of the new design as compared with the old one may be summarized as follows:

a. Reduction of 35% in construction cost, without sacrifice of strength.
b. Provision of a practical method for restoring surface in event of settlement.
c. Provision for lateral adjustment of the rail.
d. Elimination of insulation problems.
e. Elimination of reflected noise.
f. Elimination of feeling of rigidity.

Patentable features in both old and new designs have been covered by patents issued and pending.

The principal advantages of a roadbed of this type are as follows:

1. Greater safety
2. Higher permissible speed
3. Lower cost of maintaining track
4. Lower cost of maintaining equipment
5. Reduced train resistance

Of these, the reduction in the cost of track maintenance is the most tangible and possibly the most important. Aside from correcting some minor errors of design, there has been no money spent on the Pere Marquette's first installation since it was built. The cost of renewing ties and ballast is eliminated. Owing to its continuous support, the rail can be made lighter in section and much harder; thus greatly reducing the frequency and cost of rail renewals.

Track labor is almost eliminated, being reduced to inspection, rail renewal and occasional adjustment for surface. The saving in cost of maintaining track would depend on the density of traffic. A study of maintenance statistics on a number of heavy traffic railroads shows that a saving of $2000 or more per mile would be effected on the main track of such roads.

Elimination of low joints and soft spots in the track would undoubtedly result in a very considerable reduction in the cost of maintaining equipment, but this saving could not be measured unless a considerable mileage of the improved track were in use.

A comparison of train resistance on the improved track and on the adjacent track of ordinary construction was made in 1928. The resistance was computed by measuring the retarda-

tion of drifting cars and the result showed about 15% less resistance on the improved track. The methods used were rather crude, the retardation of the cars being measured by taking the time required to pass over consecutive sections of uniform length by means of stop watches. More accurate means of measurement might show a somewhat different result, but it is evident that the resistance is less on a smooth and rigid roadbed than it is on track of the usual type.

The value of greater safety and higher permissible speed cannot be measured in money; but in view of the present demand for faster movement of both passengers and freight, it is of very great importance. Combined with the tangible saving that would be effected in the cost of maintenance, it is enough to make further development of this type of track highly desirable. First class track of the usual type costs from $30,000 to $35,000 a mile and the advantages, tangible and intangible, of a permanent roadbed would appear to justify amply its additional cost of $13,000 to $18,000 per mile.

Readers are referred to the JOURNAL *for April, 1930, for discussion which may develop. Such discussion should reach the Secretary by March 1, 1930.*

Design Of Reinforced Concrete Columns Subject To Flexure

BY HARDY CROSS*

Columns of reinforced-concrete are nearly always subject to flexure and often to heavy flexure. Sometimes the flexure is a result of bracket loads but more frequently it is due to bending of connecting girders or to shrinkage or temperature changes.

The theory of stress analysis on the usual assumptions—which are not very accurate—is treated in meticulous detail in the literature, but the resulting formulas are so awkward as to be useless in design. The designer then resorts to tables, but these are necessarily very limited in their scope.

But stress analysis is only a small part of the problem. The magnitude of the moments involved is uncertain; the allowable stresses are more uncertain even for known forces. Is not a simpler design procedure which takes account of more elements in the problem to be preferred?

The object of this paper by Professor Cross is to encourage discussion as a whole of an important subject which has heretofore been treated only in a few individual aspects.—Editor

Nearly all columns of reinforced concrete are at some time subjected to a combination of direct stress and bending; most of them receive their greatest stress under such a combination; many of them receive their critical design stress under combined stress. It is the purpose of this paper to point out certain peculiar conditions which result from these combined stresses where columns are designed according to usual practice in such cases.

Our code specifies certain basic stresses for columns axially loaded, to be applied on the transformed section. Where columns

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are subjected to bending these stresses may be increased in tied columns by one-third and in spiralled columns by 15% fc′.

It is specified that "moments shall be computed and provided for." The beam theory $f = \frac{P}{A} + \frac{Mc}{I}$ is used in computing stresses, the effective section being the transformed section with concrete in tension omitted.

In only one case is the method of computation of column moments indicated. It is specified that wall columns in flat slab construction shall be designed for a total moment of $\frac{WL}{35}$ which is to be distributed between the two columns in the ratio of the values $\frac{I}{h}$. This is apparently an oversight, since the stiffness of a column is affected both by the value of E and of I; the distribution should therefore be in the ratio of $\frac{EI}{h}$ or, what comes to the same thing, of $\frac{fc'I}{h}$. There is some question as to whether I should be computed for the gross or for the net section of columns spirally reinforced. It seems more logical to use the gross section, because the deformation, except at the point of failure in case failure occurs, is determined by the gross section. The steel may be neglected in computing the moment of inertia as a measure of stiffness.

SOURCES OF FLEXURE

Concrete columns are often subjected to moments which vary greatly with the stiffness (EI value) of the column; occasionally they are subjected to moments which do not vary appreciably with the column stiffness. The moments may be due to loads eccentrically applied along the column, to flexure of girders connected to the column, to temperature or shrinkage of connecting girders, to wind or other transverse forces.

In practically every case in actual construction, so far as the writer can see, the amount of moment carried by the column depends to some extent on the column stiffness. In the case of eccentric loads and of transverse loads, such as wind, however, the variation with column stiffness is so small that for present purposes it may be neglected. Also in the case of moments due to flexure of connecting members, if the columns are very stiff compared with the other members at the joint, as in the case of columns in flat slab construction, the total moment carried by

the columns is little affected by their stiffness, though the distribution of the moment between the columns is considerably affected by their relative stiffness.

In the cases where slender columns are bent by flexure of heavy girders or by changes in length in the girders, the moments are very much affected by the stiffness of the column itself.

Problems in the design of columns subject to flexure may, then, be divided into two classes depending on whether the amount of moment to be carried does or does not vary much with the column stiffness.

BENDING MOMENTS IN COLUMNS NOT INTEGRAL WITH GIRDERS

It is assumed by some designers that if the girder is poured separately on top of the column without dowels, eccentricity in the column is thus avoided. Unless rocker bearings are used, it should be evident that up to the point where tension occurs it makes no difference whether the column is continuous with the girder or not. If tension is indicated, then the center of pressure lies somewhere between the kern point and the column edge; in this case the maximum stress intensity is twice the average on that part of the column section which is not in tension.

DESIGN METHODS

The usual method of design is to determine first the size of column for direct stress and then test this for combined stress. If the computed compressive stress is too high, several possibilities suggest themselves. We may add more steel or enlarge the column all along the axis or we may flare the column where the high moments exist or we may use stronger concrete. These methods may be effective or not depending on conditions. A study of these conditions is one of the objects of this paper.

Rectangular homogeneous columns subject to flexure about one major axis would be designed by the formula

$$f = \frac{P}{A}\left(1 + 6\frac{e}{d}\right)$$

This suggests the use of a similar formula in designing columns of reinforced concrete

$$f_c = \frac{P}{A_t}\,(1 + 6\,e/d)c$$

Where A_t is the area of the total transformed section

$$A_c + (n-1) A_s$$

c is a correction constant whose value depends on

(a) the percentage of reinforcement
(b) the eccentricity
(c) the distribution of the reinforcement between ends and sides
(d) symmetry of reinforcement
(e) embedment of reinforcement

The constant c for ordinary cases varies within rather narrow limits, from about 0.8 to 1.2. For $f_c' = 2000$ and $p = 0.03$ it varies from about 0.9 to 1.1. It is not practicable to completely tabulate the values even for all common cases. In the case of rectangular columns we have to deal with both end steel and side steel and hence have, even in symmetrical cases, three parameters for our tables, $\frac{d'}{d}$, n, and the ratio of side steel to end steel; there is always side steel. Irregular and unsymmetrical cases, though common enough, cannot be tabulated at all, nor can the very common case of flexure about two major axes.

This constant c is, in a rough way, a measure of the efficiency of the section as a column. The larger the value of c, the more deeply is the column cracked and hence only a small part of it is effective in carrying the load. We expect deep cracking where the eccentricity is large or the steel percentage is small or where n is small.

DESIGNING FOR INVARIABLE MOMENTS

Where the bending moment on the column is a fixed quantity relief of overstress can be had by increasing the column size or by increasing the amount of longitudinal reinforcement and also by increasing the strength of the concrete.

Increasing the strength of the concrete, however, is surprisingly ineffective. In the first place, for the same size of column and amount of steel, A_t decreases and c increases. In addition, the allowable stress as given by the code for spiralled columns decreases less rapidly than the strength of the concrete.

In the case of tied columns the following relative values of

A_t and of c are found for various concrete strengths when compared with 2000 pounds concrete.

$$e/d = 1 \qquad p = 0.02$$

	Relative A_t	Relative c	Relative Strength
f'_c = 2000	100	100	100 x 1.0 = 1.00
3000	92.1	120	77 x 1.5 = 1.15
4000	88.5	132	67 x 2 = 1.34

For spiralled columns we must also take account of the relatively low stresses permitted for concrete with high strength. Thus, for combined stresses the code permits for p = 0.02.

		Relative
for f_c' = 2000	f_c allowable = 960	100
3000	1290	134
4000	1620	169

If we take e/d = ½, compute the values of A_t/c and multiply by the ratio of these allowable stresses

			Relative Strength
for f_c' = 2000	100 x 100	=	100
3000	84 x 134	=	112
4000	71 x 169	=	119

In spiralled columns, then, there is almost no increase in column strength due to the use of increased concrete strength where there is very much bending. Even for axial load only, the relative strength of the spiralled column increases very slowly as the relative strength of the concrete increases.

VARIABLE MOMENT

Where the moment in the column is a result of deformation rather than of load, the moment in the column is itself a function of the size of the column and of the elastic modulus, or, what comes to about the same thing, of the strength of the concrete of which it is made. Where a girder is connected to a column, the top of the column must rotate through the same angle as the end of the girder. If the girder is very stiff compared with the column, this angle is affected only slightly by the size or stiffness of the column. The column, then, is subject to a fixed deforma-

tion and must be designed for the deformation and not for any particular moment. The process of computing the moment from the deformation and then computing the stress from the moment is a rather awkward device, justified by our greater familiarity with this process.

If the strength of the concrete is increased, the same deformation takes place. Now there is a difference between concrete and steel which is very important in this connection. Steel fails at a given stress dependent on its strength whereas concrete fails at a given strain which is practically independent of its strength. Evidently, then, there is no advantage in increasing the strength of concrete which is subject to a fixed deformation.

Evidently for a given rotation in the column, the outer fibre will be stretched in proportion to its distance from the column axis; in other words, the bending stress varies directly with the column diameter.

If the stiffness of the column is increased further, the resistance of the column restrains somewhat the bending of the connecting girder and the column stress due to bending increases less rapidly than the column diameter. If the column becomes very stiff relative to the girder, a point is finally reached where the bending stress decreased as the column diameter increases.

For stresses due to temperature change or shrinkage of the connecting girder, also, there is no advantage in using stronger concrete and there is a disadvantage in increasing the column diameter.

HIGH STRENGTH CONCRETE

In reference to the use of high strength concrete we have seen above that increased concrete strength increases the column strength very little even where the moment is fixed. In the case where the moment varies with the column stiffness, the use of high strength concrete may, if we follow the code, actually weaken the column, so that a larger column is needed the greater the concrete strength.

To take a very simple case, assume a single spiralled column connected to one end of a girder, the other end of the girder being rigidly fixed. Assume that $\frac{I}{L}$ for the girder is five times $\frac{I}{L}$ for

the column when $f_c' = 2000$. Then for other concrete strengths, the column dimensions being otherwise the same, the value of c/d will be increased because of the increased stiffness of the column. The relative factor of safety can now be computed for the column having stronger concrete. We find,

Value c/d for $f_c' = 2000$	Factor of Safety Relative to a Column in which $f_c' = 2000$ of Columns in Which $f'_c =$		
	2000 lb. # D"	3000 lb. # D"	4000 lb. # D"
¼	100%	90%	85%
½	100%	82%	73%
1	100%	79%	69%

The figures here given are based on compressive stress. Some of these columns would be overstressed in tension also.

The point illustrated is that the common practice of using in the same frame work members in which the concrete is of different strengths may be dangerous if we accept the rulings of the code.

VARIATION OF COLUMN DIAMETER

Under existing theories of column design, solutions of the problem of combined stress are so complicated that precise generalizations are hardly possible. The following statements, however, give some idea of the effect on the bending stress of variation in column diameter where the bending stress is a result of the flexure of connecting members.

If both dimensions of the column are varied, so that the column remains square or round, then when the stiffness of the column is about one-fifth the sum of the stiffness of all members at the joint, increasing the column diameter does not affect the bending stress very much. If the column is relatively stiffer than this, the stress decreases as the column size increases, slowly at first and then more rapidly until when the column stiffness approaches one-half the total joint stiffness, the bending stress is decreasing about as rapidly as the diameter increases. If the column is

relatively less stiff than one-fifth of the total joint stiffness, the bending stress will increase with increase in the column diameter. It will usually be more economical to widen the column than to make it deeper.

Columns are usually designed first for axial load, the bending in them is computed and then they are tested for compressive stress. If the stress in the concrete is found to be larger than allowable, the column size may be increased. This reduces the axial stress and may increase the bending stress or may not affect it or it may decrease it. If the bending stress is high and the column relatively slender, then the column diameter must be greatly increased. Of course if the column diameter is increased sufficiently, then the bending stress as well as the axial stress will eventually fall off, but we have now changed the construction from girder and column to a rigid frame in which the column functions chiefly to resist moment.

If the stresses due to expansion of connection girders are fairly high in the original column, they may increase so much as the diameter is increased as to make a design in which the girder is continuous with the column entirely impossible if the column is to remain square or circular and of uniform section. This is common in the design of continuous viaducts of reinforced concrete.

TENSILE REINFORCEMENT

Tensile reinforcement in columns presents what is called reversed stress. Cases of reversed stress are common; the most familiar is that of counter braces in steel trusses.

Assume that in a certain column the stress in the tensile steel due to dead load has an intensity of 10,000 lbs. per sq. in. compression and that the stress is changed by certain live load conditions to 10,000 lbs. per sq. in. tension. If the live load is doubled, the stress will be about 30,000 lbs. per sq. in. tension, or three times as much as before.

If the bending is large there seems to be less chance of any great increase in its value due to change in load conditions or to erroneous assumptions in its computation than if the moment is small. This question of factor of safety should be automatically

taken care of in the method used for computing the tensile stress. How much adjustment should be made is, however, a matter of judgment. It should be recognized that overstressing the tensile steel, while objectionable, does not threaten immediate failure.

Even an accidental increase in concrete strength may seriously overstress the tensile steel. Where the moment varies rapidly with the diameter of column, increase in the size of column without change in the amount of steel may result in overstress in the steel.

The amount of steel required for a given moment could be found simply by taking moments about the centroid of compression if we knew its position. The writer recommends that the required area of tensile steel be computed by assuming the centroid of compression at a distance 3/4d from the tensile steel and by using the usual stress in the steel.

In circular columns one-fourth of the peripheral steel is to be considered as tensile steel.

This will give designs somewhat more conservative than will an exact solution using the present code. It should be realized, however, that if the eccentricity is small, the extra amount of steel is not very important; if the eccentricity is large, an unsymmetrical design is indicated and an exact solution is rather tedious. Additional steel adds very little to the cost of the column.

In nearly all cases maximum tension in the reinforcement occurs under an entirely different condition of loading from maximum compression. Usually the maximum tension in the reinforcement is very much greater than the tension which accompanies maximum compression. Whether high unit stresses in the steel are dangerous and to what extent they are really objectionable in such cases depends on circumstances. Evidently tensile stresses exceeding the elastic limit would tend to destroy the bond but this would occur for only a short distance.

FLARED COLUMN HEADS

Where a slender column is overstressed by flexure of connecting members the economical treatment is to increase the column diameter in the region of high moment near the top of the column. This increases somewhat the moment in the column but increases

greatly the resistance to such moment. It will be found that the moment increases less rapidly than the diameter of the flared head while the stress decreases about as the square of this diameter. A very good method of design is to increase the diameter at the top in proportion to the overstress and continue the flared head down to the point where it is no longer needed for overstress.

ALLOWABLE STRESSES

On the subject of allowable stress our data are very limited. Most concrete columns receive their greatest stress from a combination of flexure and direct load and it is not improbable that they would fail under such a combination; it is unquestionable that many would fail in this way, yet test data on the strength under such conditions are almost absent from the literature.* It seems that information as to strength under combined axial load and flexure is more immediately needed than further information as to strength under axial load.

Data on the strength of columns eccentrically loaded might lead to a more dependable value of the axial strength as a limiting condition. To the writer this seems a promising line of investigation.

Consideration of the permissible stress involves the following topics:

(a) allowable axial stress with and without spirals
(b) effect of shrinkage and time flow
(c) effect of spirals in cases of combined stress
(d) general theory of flexure in cases of combined stress
(e) effect of localization of high stress on any cross section and also along the axis of the column.

It is obviously not possible to make definite statements on these points in view of the present status of the experimental data.

Where high bending stresses exist it seems doubtful whether spiralling justifies such high stresses as are permitted (1280 lbs. per sq. in. on 2000-lb. concrete with 6% of logitudinal reinforcement) even if high stresses are permissible in such columns where the stress is uniform over the section.

*For a brief review of each data see "A Study of Bending Moments in Columns," F. E. Richart, Am. Concrete Inst. *Proceedings*, Vol. 20, p. 495, 1924.

On the other hand there is considerable reason to doubt whether the theory of the cracked section gives a valid basis for computing compressive stresses in reinforced concrete columns subject to combined stresses. In these columns the shear and bond stresses are small and there is not much cracking. The theory of the cracked section implies an improbable variation in the stress conditions on the compressive face where a crack occurs on the tensile face.

DESIGN METHODS

The problem of rational design here involves, as indicated above, problems of method of analysis, of allowable stress, of factor of safety, of methods of reducing overstress. The problem is complicated in all its phases. For rational and expeditious design we need a simpler procedure than that now specified.

After considerable study of the problem, the writer suggests the following procedure as approximately satisfactory in the light of our present knowledge.

Determine the required section from

$$A_t = \frac{P}{f_a} \text{ or } \frac{1P}{2f_a} + \frac{P'}{f_b}$$

P is the axial load.

A_t is the required area of transformed section $= A_c + (n-1)\, A_5$.

P' is the equivalent flexural load $= \frac{6M}{d}$

f_a is the allowable axial stress

f_b is the allowable bending stress

The amount of tensile steel required is to be determined by taking moments about the centroid of compression, assuming $jd = 0.75$. For spiral columns, one fourth of the peripheral steel may be counted as tensile steel; for square columns with the same reinforcement in all sides this value may be taken as one-third and for rectangular columns with all steel in the ends as one-half.

SUMMARY AND CONCLUSIONS

(a) The problem of rational design of reinforced-concrete columns in flexure is unusually complicated.

(b) There are almost no test data from which to determine

working stresses. In the absence of such data, it seems best to provide for flexure on the basis of the allowable bending stress and for axial load on the basis of the allowable axial stress. In order to allow for the small eccentricities which are already provided for by the low stresses permitted for axial loads,* the axial load may be reduced one-half where combined stresses are to be provided for.

(c) A high factor of safety should be provided for the tensile steel, as in all cases of reversed stress, especially where the bending is small. It is to be noted that additional tensile steel adds very little to the total steel required in the section.

(d) The standard method of stress analysis using the transformed section has little rational and no experimental basis. It is almost impossible to apply it in cases of unsymmetrical flexure involving tension. It can be tabulated for only a very limited number of cases. For these reasons it furnishes a most unsatisfactory basis of design.

(e) Practically all reinforced concrete columns are subject to flexure, most of them receive their highest stress under combined stress, many of them would undoubtedly fail under combined stress. Too much attention seems to have been given to the stress analysis as compared with the lack of attention to determination of the moments or of the allowable stresses.

(f) Bending stresses are commonly a result of flexure of connecting members. There is, and always will be, a good deal of uncertainty about the relative stiffness of concrete members. All that we can hope to do is to provide consistently for all elements in the problem. Great precision is futile.

(g) Where overstress is indicated due to bending or expansion of connecting members in a column the size of which has been determined by axial stress only, different procedures are indicated depending on the relative amount of bending stress and relative column stiffness. If the column is flexible, increasing the diameter will increase the bending stress; if the column is stiff, increasing the diameter will decrease the bending stress. If the bending stress is very high in a flexible column, the diameter must be increased until the column is very stiff if a column of uniform

*The writer is not sure that such allowance was made in fixing allowable axial stresses. Information on this point would be valuable.

section is desired; in such a case, the most economical procedure is to increase the column diameter only in the region of high moments.

(h) The tensile steel may be designed by taking moments about the center of compression, assuming jd = 0.75 and using the usual stress in steel.

(i) The total area of transformed section required may be designed by the formula

$$A_t = \frac{P}{f_a} \text{ or } \frac{1}{2}\frac{P}{f_a} + \frac{6M/d}{f_b}$$

(j) Where double flexure occurs, add the effect of moments about the two axes.

(k) High strength concrete offers little advantage where there is much flexural stress in the column. Where a flexible column has high bending stress due to flexure of connecting members the use of high strength concrete in the column alone is definitely harmful. Similarly, abrupt changes in concrete strength in columns on successive floors is harmful.

Readers are referred to the JOURNAL *for April 1930 for discussion which may develop. Such discussion should reach the Secretary by March 1, 1930.*

Simplified Rigid Frame Design

Report of Committee 301

BY HARDY CROSS*, AUTHOR CHAIRMAN

In the preparation of this report to which Professor Cross was assigned as author-chairman, he has had the benefit of criticism and suggestions from the critic members of Committee 301, Simplified Rigid Frame Design—Clyde T. Morris, Prof. of Structural Engineering, Ohio State University, Columbus; F. E. Richart, Research Associate Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana; Albert Smith, Smith and Brown, Consulting Engineers, Chicago.

THE GENERAL subject of rigid frames could be discussed from several viewpoints. A comprehensive treatment would involve almost every phase of design and construction in reinforced concrete. It has seemed necessary, therefore, to restrict the scope of this report. The paper attempts to indicate the object of analysis of such structures and to recommend a method of analysis available for their design.

The first and the most important thing to recognize about rigid frames is that the design of these structures involves many inter-relations. Absolute precision in the solution of these problems is impossible. All that is possible and all that should be sought is a reasonably consistent basis of comparison of different structures or of different designs of the same structure, or of different parts of the same structure.

The writer of this report has discussed elsewhere the general problem of moments in continuous concrete structures.[1] It is

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Author's Note—The writer of this report wishes to thank the other members of the committee for their interest in the problem and for their helpful criticisms. He is especially indebted to Professor Richart for unusually useful suggestions.

[1] "Continuity as a Factor in Reinforced Concrete Design," *Proceedings* A. C. I., 1929, Vol. 25, p. 669.

proposed here to state briefly a method of analysis recommended for ordinary cases.

It must be recognized that it is not safe to neglect continuity as an element in concrete design, and that it is vastly more important to make some consistent provision for continuity than it is to make that provision very precise.

Here, as elsewhere in structural analysis, any effort to substitute formulas and rules for a clear conception of the principles involved will prove dangerous.

It should be recognized that continuity may involve either flexure or torsion. This means that the rotation of the end of the loaded beam may be restrained either by its connection to other beams in line with itself, which will be bent by this rotation, or by continuity with beams or slabs running normal to its own length, which will be twisted by this rotation. The torsional element has, in the past, been frequently overlooked as an element in the problem.

WHAT IS A RIGID FRAME?

The term, "rigid frame," has been used in different senses by different writers. It may be taken to apply to all frames having rigid connections at the joints and would then include buildings, bents and viaduct structures. By some it has been restricted to those structures which are made continuous with the object of improving the design, as has been done in arched bents used for industrial buildings, hangars and similar structures and also for rigid viaduct frames at highway over-crossings having limited clearance.

In some structures continuity of columns with the girders is a distinct disadvantage to the designer because it complicates the design without saving material and is justified only by its convenience to the constructor; while in others continuity is definitely depended upon to produce economy of materials. The effect of continuity on the moments and stresses is in one case similar in some ways to what is referred to as secondary stress in steel construction, while in the other the moments produced by continuity are themselves primary. In one case we do not depend on continuity to support the load; in the other case we depend definitely on continuity to support the load.

WHAT CONSTITUTES A COMPLETE ANALYSIS?

Complete analysis of a rigid frame involves curves of maximum positive and negative moments on girders, curves of maximum shear on girders, determination of maximum tension and maximum compression in all columns, and of maximum stresses from a combination of torsion and flexure in girders. Such complete studies in ordinary cases are futile and impractical. Only in the most important structures are they justified.

It must be recognized that buildings differ from viaducts and that multi-story buildings differ from single story buildings, not only in the relative importance of the problems involved but also in complications which arise from uncertainties of loading and from improbability of certain combinations of loading.

SOURCES OF STRESSES

To be comprehensive, an analysis of a rigid frame should include the effect of live load, dead load and wind, of tractive, centrifugal, and other transverse forces, of temperature change and shrinkage, and perhaps of anticipated settlement of supports.

But it must be clearly understood that a mere tabulation and summation of effects from these sources, with no attention to the relation of factor of safety to probability, is largely a waste of time. For example, we should in general be satisfied if settlement combined with other causes does not produce stresses which threaten collapse.

SCOPE OF THIS REPORT

Because of the complications involved in several aspects of the subject, it seems desirable to restrict definitely the scope of this report.

The immediate need in this field seems to be a simplified procedure in analysis of rigid frames and this paper is restricted chiefly to an attempt to present such procedure. The discussion deals only with frames composed of straight members of uniform section.

The report is further limited to those cases in which the joints are not displaced in position. Except for the effect of transverse loads, the joints may be considered fixed in position in practically all rectangular frames.

Arched bents and arches, cannot be treated easily by the methods here indicated. In any case, their design seems to constitute a separate problem.

PHYSICAL ELEMENTS

Every one knows that the moments, shears and reactions in continuous construction depend on the relative stiffness of the members.

Stiffness may be definitely defined as a physical property of the beam proportional to the moment necessary to produce unit rotation of one end of a beam supported at both ends if the other end is fixed against rotation.

We do not know very accurately the relative stiffness of concrete members, nor does it seem probable that any laboratory data will ever give us precise knowledge in this respect. For important members carrying transverse loads, uncertainty as to relative stiffness does not affect the accuracy of computed results very much. The design of such members as columns, which carry either no transverse load or very little, is often much influenced by assumptions as to relative stiffness.

The stiffness of girders in bending depends on the dimensions of the stem and of any flanges present, on the modulus of elasticity of the concrete, the stiffening effect of the merging of beams and columns, the amount and disposition of steel, bond slip, if any, and span length.

Stiffness of columns is scarcely less complicated, the chief difference being in the absence of the flange.

As to stiffness of beams in torsion, we know almost nothing even in those cases where a beam is rectangular. Obviously the twisting of a T-beam is very much restricted by the presence of a flange, and its torsional stiffness very much increased thereby. It seems probable that concrete deck beams are quite stiff in torsion.

For beams of constant section to which the beam formula $f = \frac{Mc}{I}$ can be applied, the flexural stiffness is proportional to the "stiffness factor," $\frac{EI}{L}$, where E is the elastic modulus, I the moment of inertia and L the span length.

The moment of inertia of a beam of reinforced concrete and especially of a T-beam is a rather indefinite quantity. It has

usually been recommended for rectangular members that I be taken as proportional to the product of the width by the cube of the depth. Reinforced concrete girders are, however, rarely rectangular and as to the effect of the slab of T-beams we know very little. Any assumptions on this point must of necessity be almost entirely arbitrary.

We may say, then, that the stiffness factor varies with $\frac{EI}{L}$ and write $\frac{EI}{L} \propto \frac{Ebd^3}{L} \propto EA\frac{d^2}{L} \propto \frac{EA^2}{L}\frac{d}{b}$.

For square columns $\frac{d}{b} = 1$. For beams we may assume $b = \frac{d}{2}$ for the stem and if further we assume an arbitrary multiplier 5 for the effect of the flange, we derive as approximate relations for columns and T-beams of usual proportions.

Relative stiffness of a column $= E_c\frac{A^2}{h}$.

Where (A) is the area in square inches
(h) is the height in feet.

Relative stiffness of a girder $= 10E_c\frac{A^3}{L}$

Where (A) is the area of stem in square inches
(L) is the span in feet.

These rules are highly arbitrary, but are at least useable and are perhaps as near the facts as any. The girders are assumed to be T-beams of ordinary proportions. For girders with relatively thin slabs or relatively wide stems the stiffness should be reduced somewhat, but the writer does not pretend to know exactly how much.

Mr. Albert Smith has suggested to the writer that the arbitrary multiplier used for T-beams should be two or four and also that some provision should be made for the effect of the reinforcement. Probably both points are well taken, and the second suggestion is unquestionably correct but the writer knows of no data definitely applicable to the problem. For the purposes of this report, the writer has considered simplicity more desirable than precision in determining the stiffness factor.

SIGNS OF BENDING MOMENTS

It is recommended that for bending moments in horizontal members the same convention of signs* be used as is customary in design procedure and that the same convention be applied to

*Positive moment sags the beam; negative moment hogs the beam.

columns when they are looked at from the right as a drawing is usually read.

If the moments in columns are written to the left of the columns at the top of the columns and to the right at the bottom of the columns, a joint will be balanced when the algebraic sum of the moments on the right and left sides of the joint are equal.

Thus the joints shown at the bottom of Problem 2 are balanced. The direction of curvature of the members is shown by dotted lines.

If these conventions are followed no difficulty should be found in the signs, since for given fixed-end moments in the loaded members, there is only one way in which the signs of the distributed end moments can be arranged to balance the joint—that is, to give the same algebraic total moment to right and to left of the joint.

SPAN LENGTH

All methods of analyzing rigid frames assume that the moments are computed on a length of member, center to center of intersections, and further assume that all members at any joint meet at a point. It is important to understand that by none of these methods is it possible to use directly the clear span consistently in computations.

EXACT METHOD OF ANALYSIS

The method of analysis proposed here will give accurately the moments at the supports of any frame made up of straight members of uniform section, provided the joints are not displaced by loading and provided the physical properties of the structure and the conditions of loading are accurately known.

The method to be used involves the calculation of moments at the ends of all beams in the frame under certain artificial conditions of restraint, then a redistribution of unbalanced moments by arithmetical proportion when the artificial restraints are removed. A clearer conception of the method recommended can be had by thinking of moment distribution as a physical procedure than by considering the mathematical relations involved.

Consider a continuous frame of several spans, under given loading. Denoting the junctions of members as joints, if all

joints are originally held against rotation, the end moments for each span can be found as fixed-end moments.* Suppose any one joint is now released; if the fixed-end moments at the joint do not balance each other, the joint will rotate, and since the ends of all members at that joint rotate through the same angle, the moment in each member accompanying such rotation will vary as the stiffness factor EI/L for that member, since this is proportional to the moment required to produce a unit rotation of one end of the member when the other end is fixed. Since, after the joint is released it will automatically come to equilibrium and be statically balanced, it follows that the sum of the newly induced moments equals the original unbalanced moment, and that by distributing the unbalanced moment among the members at the joint in proportion to their stiffness factors, the induced moments are determined.

In each of the members which has received its share of the unbalanced moment, there is produced a moment at the far end equal to ½ of the distributed moment at the joint end, but of opposite sign. This follows because, if a member of uniform section, supported at the ends, is fixed at the end B and subjected to a bending moment at the end A, the moment produced at B is ½ the magnitude of the moment at A, and of opposite sign. Thus the balancing of a single joint results in setting up additional unbalanced moments at the adjacent joints. The joint that has been balanced may now be held against further rotation while another joint is released and balanced in a similar way. The amount of unbalance is thus rapidly reduced at all joints, and the process is continued until the amount of unbalance remaining is made as small as desired. It is evident that the process may be carried to any desired degree of precision. If approximate results are sufficient, the process of distribution need be repeated only a few times. The various moments found and combined may be thought of as the terms of a rapidly converging series, for which the number of terms employed determines the precision of the result.

The above operations may now be systematized and the application of the process stated in a few words.

*The derivation of formulas for moments at the ends of "fixed" spans may be found in most texts on Mechanics.

1. Compute the moments due to the given loading which would exist at the ends of each member of the frame if these ends were fixed, or held against rotation.

2. At each joint distribute the unbalanced moment (the difference between the fixed-end moments for the members meeting at the joint) among the members at the joint in proportion to their stiffness factors, EI/L.

3. For each member which has received a certain distributed moment at a joint, apply at the far end of the member a moment of opposite sign and one-half the magnitude of the distributed moment. These may be called "carry-over" moments.

4. If the carry-over moments at any joint are now unbalanced, again distribute these unbalanced moments and apply the carry-over moments as described in paragraphs 2 and 3 above.

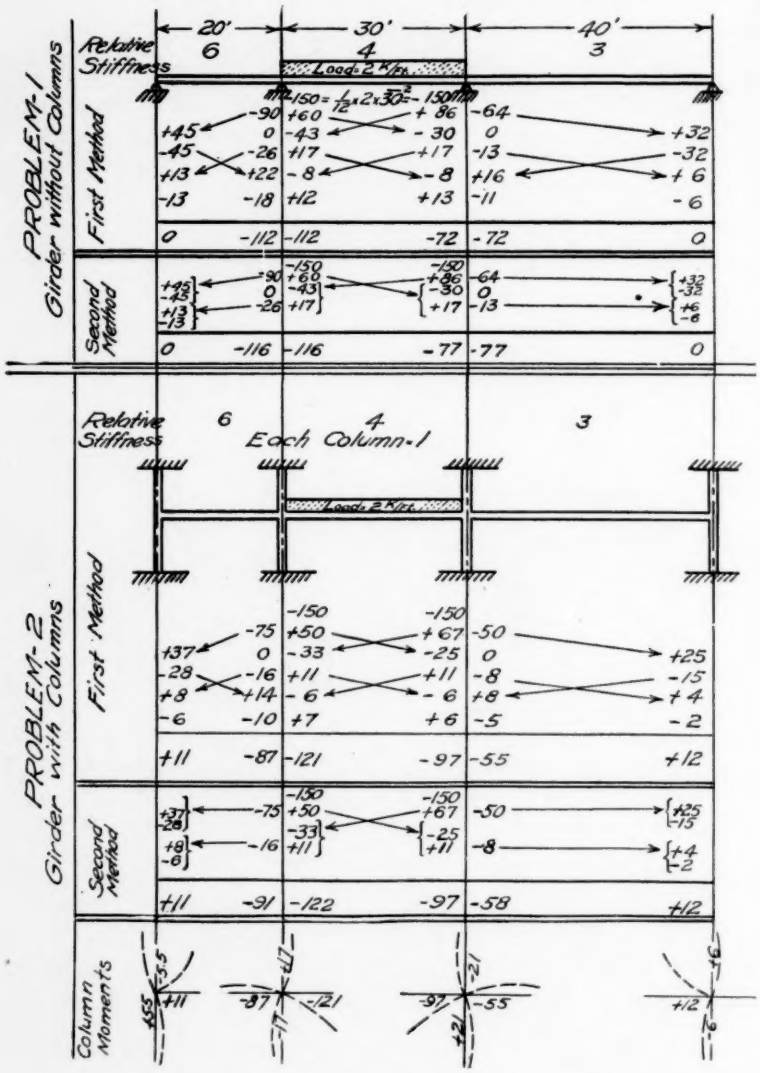
5. Repeat the process until the desired degree of precision (freedom from unbalanced moments) is attained. At each side of each joint add the original fixed-end moments, the distributed moments, and the carry-over moments. The sum is the true moment at the end of the member at the joint.

After the end moments on the members of the frame are known, all other quantities—moments at points other than the ends, shears, reactions—may be computed from statics.

ILLUSTRATIVE PROBLEMS

Problem 1—As an illustration of the application of the above method to a particular case, assume a continuous girder of three spans, with no restraint at the ends, as shown. The span lengths are successively 20, 30 and 40 ft. Assume the relative stiffness successively as 6, 4 and 3, which is correct if the girder is of constant section and material. Assume the second span only to be loaded with a distributed load of 2,000 lb. per foot of length. The fixed-end moments for this span are $WL/12 = 150{,}000$ ft. lb., and are the unbalanced moments at the ends of the middle span. For convenience, the moments will be expressed in ft.-kips, or thousands of ft.-lbs.

At the second support, distributing the unbalanced moment of 150 between the first and second spans in the ratio of 6 to 4, we have 90 on the left of the joint and 60 on the right of the joint. At the third support, distributing the unbalanced moment of 150 between the second and third spans in the ratio of 4 to 3, we have 86 on the left and 64 on the right of the joint. The signs of



these moments are so applied as to make the sums of the moments to right and left of the joint equal, or to balance the joint.

Now considering the first span the moment of −90 at the right end produces a carry-over moment of +45 at the left end. Similarly, carry-over moments

are written for the ends of each span giving successive values of +45, 0, –43, –30, 0 and +32.

These carry-over moments now produce unbalance at all of the joints, and the process of distributing unbalanced moments must again be carried through. At the first and fourth supports we have assumed free ends of beam, hence moments of–45 and –32, respectively, must be added at these joints to produce balance (the zero moment assumed).* The six distributed moments then become –45, –26, +17, +17, –13 and –32. The six carry-over moments (½ of distributed moments at opposite end) are +13, +22, –8, –8, +16 and +6.

In distributing these moments, it is seen that the unbalanced moment at the second support is the algebraic difference between +22 and —8, or 30. This 30 is distributed in the proposition 18 and 12, with signs assigned which will produce balance at the joint. A similar distribution is made at the third support. The six distributed moments now are –13, –18, +12, +13, –11 and –6.

This procedure of distribution may be carried as far as seems desirable. If we stop at this point and add values we find end moments as follows: 0, –112, –112, –72, –72 and 0. In any case it is obvious that the procedure should be stopped after unbalanced moments have been distributed to balance the joints. Had we stopped after the first distribution, the moment at the second support is seen to be –90, after the second distribution it was –116, after the third, –112. If a fourth and fifth distributions were carried through, the values found would be –113 and –113.5. The exact value for this moment is –113.36. The third distribution is seen to give quite satisfactory precision in this case.

Problem 2—Assume the same beam and loading, but with beam supported on and rigidly connected to columns. The relative stiffness of each column is taken as 1.

The same procedure is followed as in the previous case. The distribution of moments will take into account the share of unbalanced moment to go to each column, but it will be more convenient to tabulate only the moments in girders, the moments in the columns to be found later by subtraction of girder moments at each joint.

The fixed-end moments are the same as before.

At the left end of the loaded span the fixed-end moment is distributed to the four connecting members in the ratio 6, 1, 1, 4 or 6/12 150 = 75 to the girder on the left, 4/12 150 = 50 to the girder on the right. (The joint appears not to be balanced because the column moments have not been written.)

At the right end of the loaded span, the fixed-end moment is distributed to the four connecting members in the ratio 4, 1, 1, 3, or 4/9 150 = 67 to the girder on the left of the joint and 3/9 150 = 50 to the girder on the right of the joint.

Now carry over one-half of these moments with opposite signs.

Distribute again. At the left end distribute in the ratio 1, 1, 6 or 6/8 37 = 28 to the girder. At the next joint distribute in the ratio 6, 1, 1, 4 or 6/12 33

*Note that there is nothing special about this procedure; the unbalanced moment is distributed among the connecting members, but there is only one member.

= 16 to the girder on the left and 4/12 33 = 11 to the girder on the right. At the next joint 4/9 25 = 11 is distributed to the girder on the left and 3/9 25 = 8 to the girder on the right. At the right end distribute in the ratio 1, 1, 3 or 3/5 25 = 15 to the girder.

Again carry over and again distribute. In this case at the first joint from the left we distribute 14 + 6 = 20 to the four connecting members. At the next joint we distribute 8 + 6.

Add all figures in each vertical column. The final end moment are 11, 87, 121, 97, 55, 12.

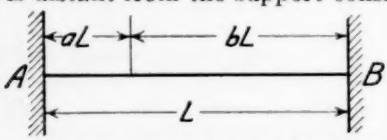
The column moments are, for the two columns at the left end, 11, for the next pair of columns 121 − 87 = 34, for the next pair 97 − 55 = 42 and for the pair at the right end 12. These moments are in each case distributed equally between the two columns in the pair because they are equally stiff. In any other case they would be distributed in proportion to the column stiffness. The signs of the column moments and the direction of curvature in the columns are as shown by the sketches.

FIXED-END MOMENTS

The fixed-end moment for beams of constant section and for uniform load is 1/12 WL where W is the load on one panel. Where the load is applied through floor beams, the fixed-end moment is C1/12 WL, where

for one intermediate floor-beam, C = 3/4
for two intermediate floor-beams C = 8/9
for three intermediate floor-beams, C = 15/16

For concentrated loads, the fixed ended moment at one end may be computed from the formula $M' = a^2bPL$, where "a" is the fraction of the span by which the load is distant from the other support, and "b" is the fraction of the span by which the load is distant from the support considered—(a + b = 1)



$M_a = ab^2\ PL$
$M_b = a^2b\ PL$

For a series of concentrated loads, the end moments should be separately computed and added.

RECOMMENDED APPROXIMATE METHODS

The method just indicated, though a method of successive approximation, is in no sense an approximate method because we can obtain by it results in which the error is made less than any assigned quantity.

Attempts at great precision in this work are usually futile, as explained elsewhere; consequently, the procedure can be conveniently abbreviated. This can be done by distributing only a few times as explained above. This method is simple but is not quite definite because we cannot specify how many distributions will be satisfactory.

Another approximate method which is perfectly definite and is usually about as accurate as is a few distributions in the method explained above is to distribute first at the unbalanced joints and then to carry-over and distribute always *away* from that joint, omitting to carry over the moments back toward the joint from which they originate.

Thus in the first problem solved above we follow the same procedure as before in distributing the original unbalanced fixed-end moments, and also in carrying over these moments and distributing the moments carried over so that the first three lines of the solution after writing the fixed-end moment are identical in the two methods. When these distributed moments are carried over, however, they are carried over in the same direction as before. The values +17 and +17 are, then, not carried over to give —8, —8 nor are the end values —45 and —32 carried over to give +22 and —16.

In the second illustrative problem, using this second method of approximation, the first three lines after the fixed-end moments are again identical with those previously found but the values +11, +11 are not carried over (carried back) nor are the values —28 and —15 carried over.

This second method of approximation is that given in the writer's paper "Continuity as a Factor in Reinforced-Concrete Design" before referred to. It is not as convenient as the first method where several spans are loaded. Maximum shears, moments and reactions, however, are most readily determined by combining results from loading individual spans. In this case the second method is rapid, can be readily condensed, and is sufficiently accurate for most purposes.

CONCLUSIONS

Experience has shown that the method here presented for determining moments at the joints of a rigid frame is easy to

remember and rapid to apply. It is the writer's belief that it will be found convenient in design offices.

A rigid frame in which all the elements of distortion are taken into account is undoubtedly better designed than one in which arbitrary coefficients are used. This, however, does not seem to justify abrupt departure from existing practice in design specifications.

But considerations of continuity will be more and more forced on the attention of design offices. It is, therefore, recommended that our specifications be revised so as to make it permissive to use moment coefficients derived from elastic analysis in all cases provided adequate provision is made for the elements depended on in such analysis. Thus, where the stiffness of columns is depended on in computing girder moments, adequate provision must be made for the moments resulting in the columns, or where torsion of connecting members is depended on, adequate provision must be made for such torsion.

Evidently such a recommendation will ultimately require also workable recommendations as to what constitutes adequate provision for flexure in columns or for torsion in beams, but anyone who will consider the present status of design practice in rigid frames should see that the exact details of such provision are less important than is some method of encouraging designers to recognize the facts and to make any provision at all for them.

Such a change in specifications should result in appreciable reduction in the positive steel requirements in buildings. In the writer's opinion such reduction is perfectly proper where its basis is clearly recognized. New possibilities of economy will be opened to the skilled designer.

In considering the advisability of such specifications—especially from the viewpoint of building departments or of unscrupulous designers—it should be recognized that it simply makes permissive in the cases which are perfectly regular (Sections 708 and 709 of the Building Regulations)* that which is now permitted by the specifications in all other cases. (Section 710 of the Building Regulations.)

*"Tentative Building Regulations for Reinforced Concrete," *Proceedings* A. C. I., 1928, Vol. 24, p. 791.

It has been suggested that curves be drawn giving coefficients for varying column stiffness similar to those now in use for cases of equal spans. The writer has not thought the idea very practical at this time. The present system of coefficients does not distinguish between live and dead load, and does not use the span length between centers of intersection of the members. Such curves as proposed would, then, involve four variables—ratio of dead to live load, ratio of girder stiffness to column stiffness, ratio of clear span to span length between intersection of members, and the moment coefficient. The difficulties involved in presenting curves having four variables are very great.

There exists some difference of opinion among designers as to what combinations of loads should be considered in determining maxima, especially in regard to the weight to be given in cases of split loading. This question at least should be settled before any attempt is made toward further standardization.

Readers are referred to the JOURNAL *for April, 1930 for discussion which may develop. Such discussion should reach the Secretary by March 1, 1930.*

A Study of Portland Cement Mortars Having Diatomaceous Earth as an Admixture*

BY JESSE E. BUCHANAN†

There are few subjects presenting more insistent challenge to the serious investigator than the use of "admixtures"—and few such on which more meagre authentic data are to be had. If the man in the field feels he must know the answer he often investigates just far enough to satisfy himself for an immediate occasion and there he is likely to leave the general problem dangling. In proposed Specification 502 (Concrete Work on Ordinary Buildings)"‡ article 10 provides: "Nothing shall be added to the essential ingredients of concrete" etc. The author of the specification in his discussion of it clearly implies a negative attitude in the absence of more definitely reassuring data. All serious contributions to the subject are therefore welcome.—Editor.

INTRODUCTION

The purpose of this study is to compare the behavior and characteristics of portland cement mortars containing small percentages of diatomaceous earth with identical plain portland cement mortars. Powdered admixtures such as diatomaceous earth are used to secure uniform, workable mixtures of concrete which may otherwise tend to be harsh. Likewise, in mixtures of dry consistency, workability has in many cases been secured by the use of admixtures. Also, the segregation of ready-mixed concrete during transportation has been in some measure eliminated by the addition of small amounts of fine, inert materials such as diatomaceous earth and hydrated lime.[1]

Considerable study has been given to the quality and characteristics of concrete containing admixtures as compared with plain concrete. Strengths have been compared probably more than anything else. Yield, permeability, absorption, expansion, and

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‡Journal A. C. I., Nov. 1929, p. 1 (A. C. I. *Proceedings*, Vol. 26)
[1]Bibliography, references 2, 3, and 9.

contraction, are other items which have been variously compared. The conclusions reached have not in all cases been consistent. It is important that these comparative studies be made and that some agreement be found. If, while these admixtures are functioning to improve uniformity and workability, they have corresponding detrimental effects, these effects must be known and evaluated.[2]

This paper covers a comparison between plain mortar and mortars containing small percentages of diatomaceous earth as to yield, strength, and volume changes under various conditions of curing. The effects of diatomaceous earth on consistency and time of set have also been noted. Fine and coarse sands were used in proportions of 1:2, 1:3, and 1:4 by weight. The comparative mixes were all made with a constant water-cement ratio, and a wet, almost fluid, consistency approximating the condition of mortar used in structural concrete. Comparisons were made on mortar instead of concrete in order to eliminate the variables which would be introduced by gravel or crushed rock. Comparative results, and not absolute values, were desired.

DEFINITION OF TERMS AND REVIEW OF PROBLEM

Yield—The term yield is used here to mean the volume of mortar resulting from a mix. Two values of yield for each mix were obtained. One was computed by assuming the volume of mortar produced to be equal to the sum of the absolute volumes of its constituents; the other value was determined by measurement.

Strength—Compressive strength was determined in pounds per square inch on 2″ x 4″ mortar cylinders.

Consistency—In general, this term has been considered as indicating the fluidity or plasticity of a mix as determined by the spread or flow on a flow table.

Workability—Workability has been regarded as a quality apart from consistency although it is realized that to some extent workability depends upon and varies with consistency. Lack of harshness, internal smoothness, uniformity, and ability to work and handle with ease are the properties which differentiate one mix from another as to workability.

In a study of this kind, there are many variable factors, each of which may exert more or less influence upon the results obtained. In order to compare findings of independent studies, the results of each study should be given in terms of the materials

[2]Bibliography, references 1, 2, 3, 4, 6, and 9.

used, the methods employed, and the conditions maintained. General conclusions may then be drawn with some degree of certainty and each study will have contributed in some measure to the facts supporting such general conclusions. It is anticipated that the results obtained in these tests will be considered only in terms of the materials, methods, and conditions pertaining thereto.

While it is true that many variables enter into the production of mortar and concrete, both in the field and laboratory, there have been noted some phenomena which are always present.[3] Portland cement mortar shrinks in volume as the setting process takes place. This may be termed the first phase of shrinkage. After the mortar is set, if it then is allowed to dry out, additional shrinkage takes place; this is called the second phase. The discussion of volume changes in this paper will be entirely concerned with those changes which take place after final set has been reached. Temperature has been held constant, and will not be considered as a factor. The point of reference for all measurement of changes is an initial reading taken after 24 hours curing in a moist closet at 70° F. With this point of reference in mind, it has been noted that mortar subsequently allowed to dry out undergoes shrinkage. Mortar continually immersed in water expands with respect to the reference point named. Alternate wetting and drying is accompanied by expansions and contractions. Mortar specimen taken from structures many years old have been found to respond to wetting and drying. In general, the following interrelated factors influence quantitatively the amount of unrestrained volume change which, for given conditions of exposure, may take place:

1. Character of materials
2. Richness of mix
3. Water-cement ratio of mix
4. Size of specimen

It is extremely desirable that volume changes in concrete be reduced to a minimum. While it may be granted that diatomaceous earth as an admixture serves to improve workability, uniformity, and impermeability, the question arises as to its influence, if any, on volume changes. It is important that this

[3]Bibliography, references 1, 5, 7, 8, and 10.

question be answered correctly if possible. Some results have already been reported, however, they are not altogether in accord.[4]

MATERIALS AND METHODS USED

The materials used in these tests represent the average quality used in construction in the Moscow region:

Portland cement—A well known brand of cement was used in all mixes. Sufficient quantity for the entire series was carefully mixed and prepared in advance. The physical characteristics determined according to A. S. T. M. C 9—26 were as follows:

Specific Gravity	3.12
Fineness	11.4% retained on 200 mesh
Time of set	
Initial	3 hrs. 10 min.
Final	5 hrs. 40 min. Gillmore needles
Soundness	O. K.
Normal Consistency	24%
Tensile Strength	
7-day	275 lb./sq. in.
28-day	380 lb./sq. in.

Sand—Two kinds of sand were used, one coarse and the other fine. The coarse sand was obtained from glacial deposits at Spokane, Washington. This sand is widely used throughout the Inland Empire in concrete construction.

Specific Gravity	2.67
Silt and Clay	2.0%
Strength Ratio	
7-day	121.7%
28-day	117%
Organic content	A. P. No. 1
Fineness modulus	3.49

The fine sand was obtained from Snake River deposits at Lewiston, Idaho.

Specific Gravity	2.70
Silt and clay	3.0%
Strength ratio	
7-day	101%
28-day	107.2%
Organic content	A. P. No. 1
Fineness modulus	2.51 well graded from No. 8 down.

Diatomaceous Earth—The diatomaceous earth was obtained from a deposit in Oregon and represents the average of this material on the market as a commercial product. It is a very pure diatomaceous earth.

Retained on 200 mesh 7.2%

[4]Bibliography, references 1, 6, and 7.

TABLE NO. I

SCHEDULE AND IDENTIFICATION OF MIXES

Series No.	Sand	Proportions by Weight	Mix No.	Diatomaceous Earth. Added % by Weight of Cement
I	Coarse	1:2	1 2 3 4	0 1 2 4
II	Coarse	1:3	1 2 3 4	0 2 4 6
III	Coarse	1:4	1 2 3 4	0 4 8 12
IV	Fine	1:2	1 2 3 4	0 1 2 4
V	Fine	1:3	1 2 3 4	0 2 4 6
VI	Fine	1:4	1 2 3 4	0 4 8 12

Table I shows the number and identification of tests made. Each series consisted of four mixes which differed from each other only in the amount of diatomaceous earth added. The quantity of admixture is expressed as a per cent by weight of the cement in the mix and was added to the mix. Each mix number 1 of every series was made to the same wet consistency as measured on a flow table. Mix numbers 2, 3, and 4 of each series were then made with the same amount of water as mix number 1 of that series. This means that the water-cement ratios for the mixes in a series are constant, but the water-cement ratios of the six series differ.

All proportions were measured by weight. Mixing was done in a shallow pan in a standard manner, first dry, and then wet. Flow measurements were taken as the average of three spreads immediately after mixing was completed. Next, a one-tenth cu. ft. measure was filled, weighed, and checked. This served as the basis for measured yield. Specimens were then prepared. From each mix the following test pieces were made:

6—2″ x 4″ cylinders
4—1″ x 1″ x 10″ bars
1—time of set pat.

Every possible precaution was taken to hold and exactly duplicate all operations, readings, and conditions.

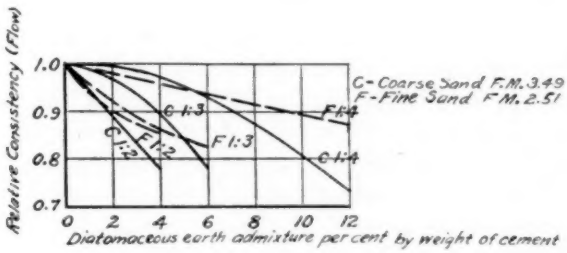

FIG. 1—CURVES SHOWING EFFECT ON CONSISTENCY OF MORTARS, AS MEASURED BY FLOW, OF THE ADDITION OF SMALL PERCENTAGES OF DIATOMACEOUS EARTH. ALL PLAIN MIXES WERE MADE TO THE SAME WET CONSISTENCY

Brass contact measuring plugs were installed in the ends of the bars. All specimens were cured in molds under identical conditions for 24 hours. Readings on time of set pats were taken with Gilmore needles in the same manner as taking time of set on cement.

After 24 hours curing, the specimens were removed from the forms. Three cylinders of each mix were placed in water to remain until tested at 28 days. The other three were placed in damp sand for 13 days after which time they were allowed to cure in air until tested at 28 days.

Initial readings of length were taken on bars after removing from molds at the expiration of 24 hours. One bar was placed in water. The second bar was left in air in the laboratory. Bar

number three was kept alternatively 14 days in water and 7 days in air while bar number four was kept 14 days in air and 7 days in water.

Expansion and contraction readings were taken by means of a micrometer screw reading, directly to 0.0001 inches. Contact was determined by means of a telephone receiver in series with the plugs in the ends of the bars. All readings were taken at 70° F. Compression tests were made in a 50,000 lb. Riehle machine at a cross-head speed of 0.05 in. per min.

RESULTS

Consistency—Fig. 1 shows the comparative reductions in consistency (flow), as the percentage of diatomaceous earth is increased. All plain mixes, 0% admixture, were made to the same wet consistency which means, for example, that more water was required in the 1:3 fine sand mix than was required in the 1:3 coarse sand mix. The figure shows that for lesser amounts of admixture, the coarse sand mixes had less reduction in consistency than did the corresponding fine sand mixes. However, for the larger percentages of diatomaceous earth, the coarse sand mixes were drier than the corresponding fine sand mixes.

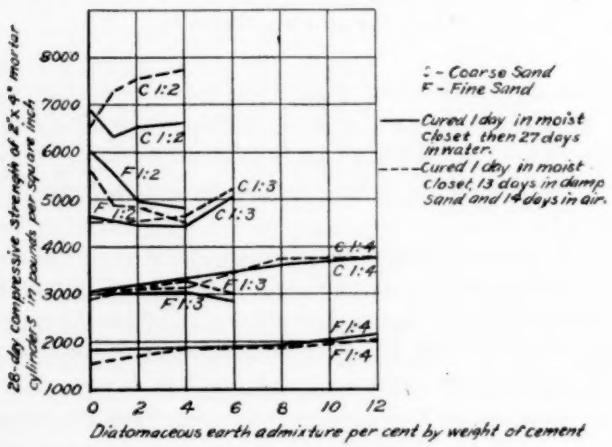

FIG. 2—COMPRESSIVE STRENGTH OF 2 X 4 IN. MORTAR CYLINDERS

TABLE NO. II

RATIO OF MEASURED YIELD TO COMPUTED YIELD

Series No.	Proportion by Weight	Mix No.	Per Cent Diatomaceous Earth	Yield Ratio
		Coarse Sand		
I	1:2	1	0	1.008
		2	1	1.009
		3	2	1.012
		4	4	1.014
II	1:3	1	0	1.018
		2	2	1.019
		3	4	1.020
		4	6	1.021
III	1:4	1	0	1.022
		2	4	1.025
		3	8	1.030
		4	12	1.033
		Fine Sand		
IV	1:2	1	0	1.005
		2	1	1.006
		3	2	1.007
		4	4	1.014
V	1:3	1	0	1.017
		2	2	1.026
		3	4	1.028
		4	6	1.036
VI	1:4	1	0	1.027
		2	4	1.032
		3	8	1.041
		4	12	1.042

Yield—Table II shows the ratio of measured yield to computed yield for each mix. As previously stated, the volume of mortar of each mix was computed from the weights of materials combined on the assumption that such volume equalled the sum of the absolute volume of sand, cement, water, and diatomaceous earth. A value of this volume was also obtained by direct measurement. Knowing the total weight of materials combined in a mix, the total volume of this combination was determined

from the weight of mortar contained in a one-tenth cu. ft. measure.

The possibility of experimental errors entering into these determinations is rather high. Since absolute values are not desired, comparative results are shown. The table shows that in all cases the measured volume is greater than the computed volume. It also shows that in any one series the ratio increases with the amount of admixture present.

These comparisons might be interpreted to mean that there is a slight increase in volume of mortar above that due to the absolute volume of admixture added. This might be termed "puffing." It is to be noted that this puffing is not due to water, because the amount of water in each mix of a single series has been held constant.

Strength—In Fig. 2 are shown the results of compressive strength tests on 2″ x 4″ mortar cylinders. The average of three cylinders was taken as the strength of a mix. Since each series was made with a different water-cement ratio with mixes of fine sand having the larger values, it was expected that coarse mixes would show higher strengths.

With the exception of Series I, (coarse sand 1:2), and Series IV, (fine sand 1:2), there appears a general trend to slightly increased strength with an increase of admixture for both methods of curing. Series I and IV are neither consistent with each other nor with the other four series.

Table III shows the unit weights of the various mixes as determined from weighing and measuring compression specimens immediately before testing.

Expansion and Contraction—Fig. 3 shows the lineal expansion of 1″ x 1″ x 10″ mortar bars cured continuously in water. The changes in length are based on initial readings taken after 24 hours in moist air. Rich mixes have a greater final expansion than do lean mixes. Coarse sand bars show greater expansion than fine sand bars.

There seems to be no definite tendency which might be attributed to the diatomaceous earth admixture. It is not possible

TABLE NO. III

UNIT WEIGHT OF MORTAR. DETERMINED FROM COMPRESSION SPECIMENS JUST BEFORE TESTING

Series No.	Proportion by Weight	Mix No.	Per Cent Diatomaceous Earth	Curing: 24 hrs. moist air; 27 days water — Unit Weight lbs./cu. ft. (Specimens Saturated)	Curing: 24 hrs. moist air; 13 days damp sand; 14 days air — Unit Weight lbs./cu. ft. (Specimens Dry)
I Coarse Sand	1:2	1	0	147.5	141.9
		2	1	147.2	142.6
		3	2	145.6	140.3
		4	4	146.1	143.0
II Coarse Sand	1:3	1	0	142.0	137.7
		2	2	142.4	136.3
		3	4	144.6	135.6
		4	6	145.0	138.6
III Coarse Sand	1:4	1	0	139.4	132.0
		2	4	140.2	132.4
		3	8	140.9	132.5
		4	12	141.6	132.7
IV Fine Sand	1:2	1	0	142.9	134.8
		2	1	143.2	134.8
		3	2	142.8	134.6
		4	4	141.6	130.6
V Fine Sand	1:3	1	0	141.8	132.2
		2	2	140.9	132.0
		3	4	141.4	130.6
		4	6	140.3	131.1
VI Fine Sand	1:4	1	0	136.8	125.9
		2	4	139.3	127.4
		3	8	137.6	124.8
		4	12	139.1	127.0

to conclude that diatomaceous earth increases or diminishes the expansion under water. It may, therefore, be said that no particular effect on the expansion of these mortar bars continuously immersed in water is produced by the percentages of admixture used.

While there are some inconsistencies shown in Fig. 3, there is no trend indicated. It is expected that concrete made of these mortars would probably show less final unit change. The differ-

ences between plain and admixture concrete would probably be less marked than they are in the case of these mortars. Variations of uniformity of concrete produced in the field might be expected to cause differences as much or more than those found in Fig. 3.

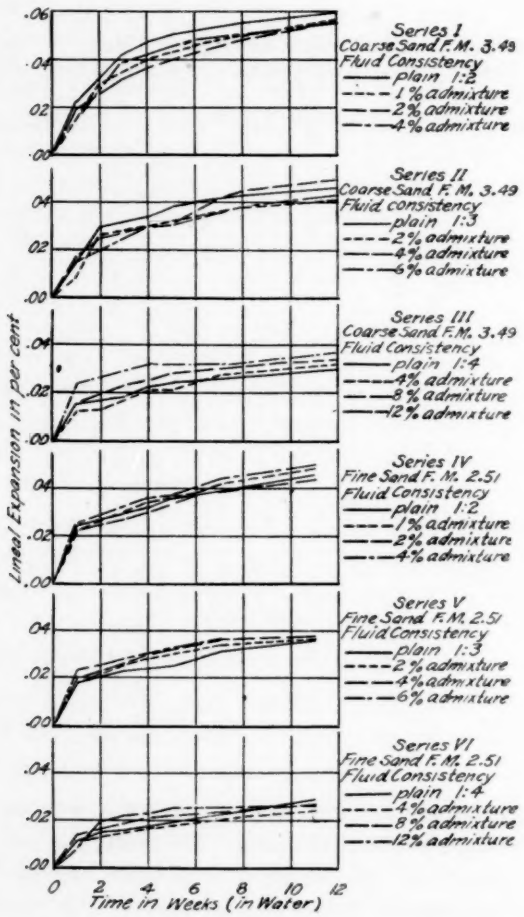

FIG. 3—EXPANSION OF 1 X 1 X 10 IN. PORTLAND CEMENT MORTAR BARS, PLAIN AND CONTAINING SMALL PERCENTAGES OF DIATOMACEOUS EARTH, CURED IN WATER AT 70° F.

Fig. 4 shows graphically the lineal shrinkage taking place in 1″ x 1″ x 10″ mortar bars cured continuously in air. As in the case of Fig. 3, no definite trend is apparent. There are some experimental variations, but it does not appear that diatomaceous

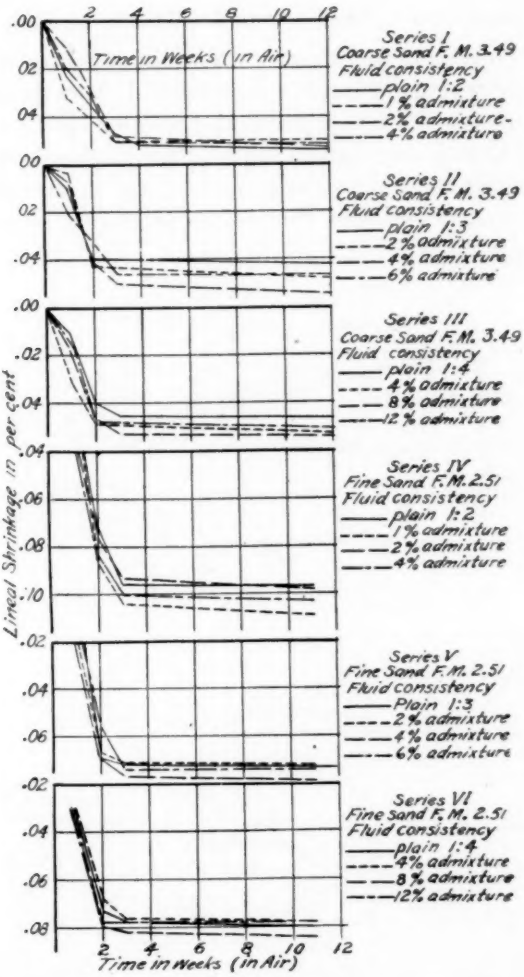

FIG. 4—SHRINKAGE OF 1 x 1 x 10 IN. PORTLAND CEMENT MORTAR BARS, PLAIN AND CONTAINING SMALL PERCENTAGES OF DIATOMACEOUS EARTH, CURED IN AIR AT 70° F.

earth used as an admixture in these mortars definitely causes a greater or lesser unit change under this condition of curing.

Absolute values of shrinkage are not of particular importance in this case, because all mortars are inherently different and have corresponding values of final shrinkage. The comparative results are considered to be the important ones.

In Fig. 5 is shown the extent of linear change using coarse sand for cycles of curing, 14 days in water and 7 days in air. Fig. 6 shows the results for fine sand mortar with the same treatment. Heavy full lines in the figures indicate the changes taking place in plain mortar bars. In order to make the diagrams more clear, only the extreme limits of change are shown for the bars containing the admixture.

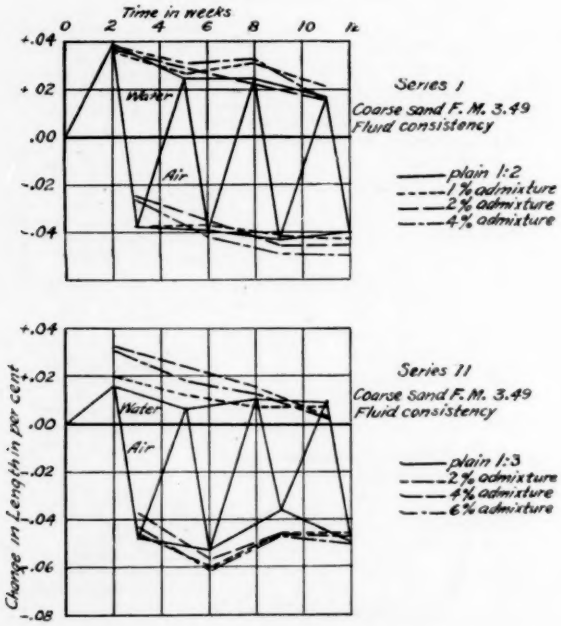

FIG. 5—SHRINKAGE AND EXPANSION OF 1 X 1 X 10 IN. PORTLAND CEMENT MORTAR BARS, PLAIN AND CONTAINING SMALL PERCENTAGES OF DIATOMACEOUS EARTH, CURED 14 DAYS IN WATER AND 7 DAYS IN AIR AT 70° F.

The total change in length for a cycle seems to be approximately the same for all bars of a series with the exception of Series II. The control bar of Series III was lost and the results for that Series have been omitted from Fig. 5. It is strikingly

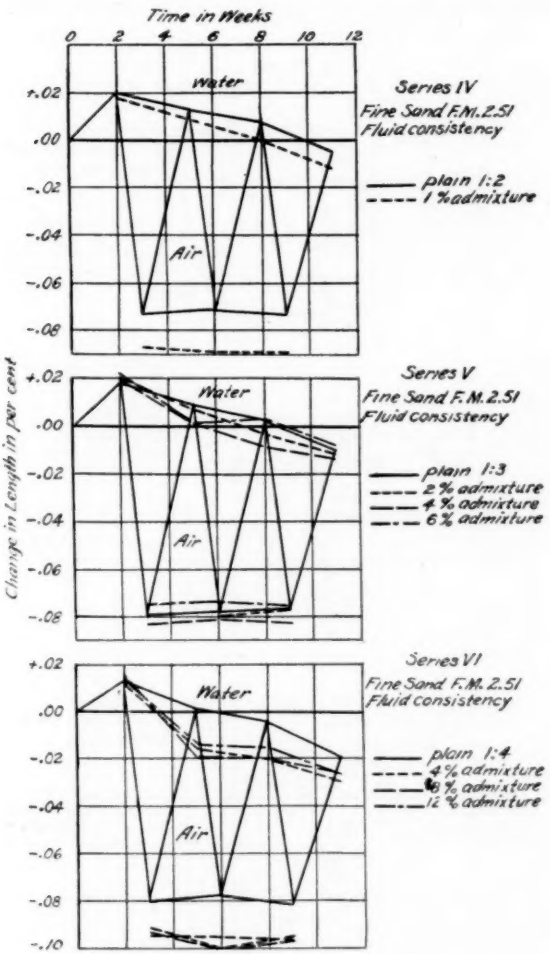

FIG. 6—SHRINKAGE AND EXPANSION OF 1 x 1 x 10 IN. PORTLAND CEMENT MORTAR BARS, PLAIN AND CONTAINING SMALL PERCENTAGES OF DIATOMACEOUS EARTH, CURED 14 DAYS IN WATER AND 7 DAYS IN AIR AT 70° F.

shown that shrinkage during 7-day air curing greatly overbalances expansion in 14-day water curing for all but Series I.

Fig. 7 and 8 show similar results. The cycle consisted of 7 days in water and 14 days in air. The amplitude of change for a series seems to be about the same except for Series III. This is a

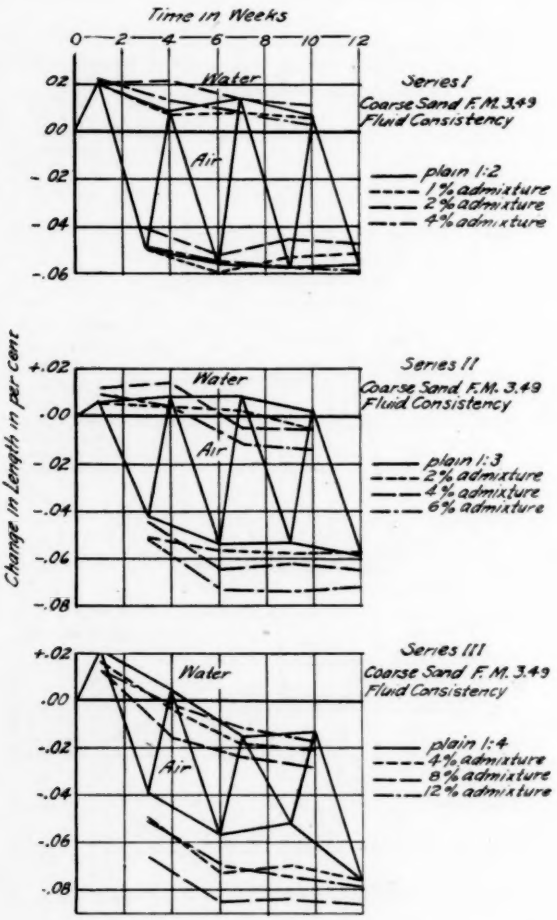

FIG. 7—SHRINKAGE AND EXPANSION OF 1 x 1 x 10 IN. PORTLAND CEMENT MORTAR BARS, PLAIN AND CONTAINING SMALL PERCENTAGES OF DIATOMACEOUS EARTH, CURED 7 DAYS IN WATER AND 14 DAYS IN AIR AT 70° F.

1:4 coarse sand mortar. The amplitude of the plain bar is much less than the total change of the bars containing admixture. However, the three bars containing respectively 4, 8, and 12 per

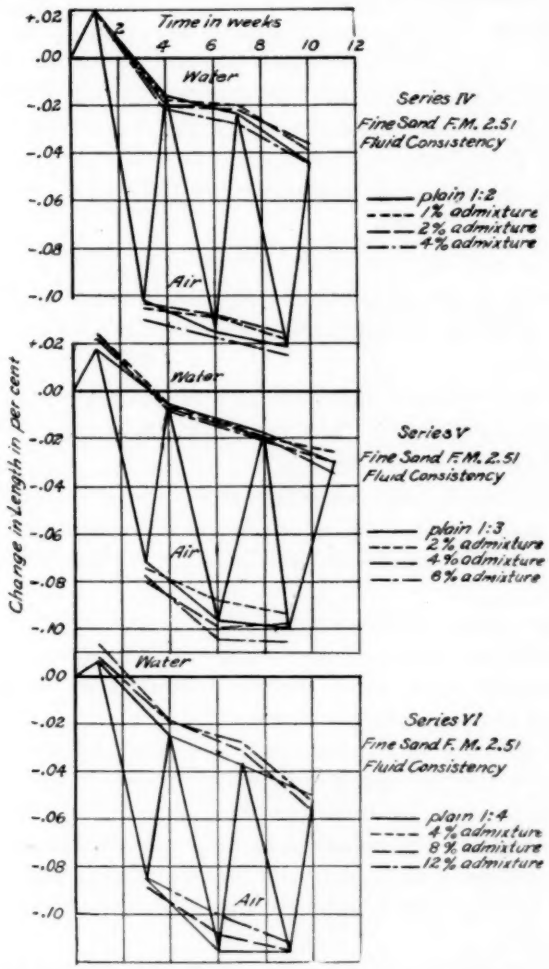

FIG. 8—SHRINKAGE AND EXPANSION OF 1 X 1 X 10 IN. PORTLAND CEMENT MORTAR BARS, PLAIN AND CONTAINING SMALL PERCENTAGES OF DIATOMACEOUS EARTH, CURED 14 DAYS IN AIR AND 7 DAYS IN WATER AT 70° F.

cent of diatomaceous earth have approximately the same amplitude. The plain bar may be defective.

Under this type of curing the shrinkage in air is much greater than the expansion in water.

Time of Set—In all cases the time of set for mortars containing admixture was less than for plain mortars. In general, a greater percentage of admixture was accompanied by a decreased time of set. While this was true for mortars, the shortening for the conditions of these tests was not of such magnitude as to hamper or limit the usefulness of mortars containing admixture as compared to plain mortars.

CONCLUSIONS

In terms of the methods, conditions, materials, and experimental variations of these tests a few general statements may be formulated:

1. With a constant water-cement ratio, the consistency of a mortar as expressed by flow decreases as the per cent of diatomaceous earth is increased.

2. The use of diatomaceous earth as an admixture in these mortars causes a puffing, or an increase in volume of mortar, which is slightly more than the absolute volume of admixture added.

3. The compressive strength of 1:3 and 1:4 (by weight), mortars containing small percentages of diatomaceous earth is approximately equal to or slightly greater than identical plain mortars of the same water-cement ratio.

4. No appreciable nor consistent difference in expansion when stored in water, or contraction when stored in air is noted in these tests between plain mortar bars and mortar bars containing small amounts of diatomaceous earth as an admixture.

5. Shrinkage in air is generally larger than expansion in water for these mortars when subjected to alternate storage in water and air.

6. The shortening in time of set of these mortars increases with the amount of diatomaceous earth used.

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Readers are referred to the JOURNAL *for April 1930, for discussion which may develop. Such discussion should reach the Secretary by March 1, 1930.*

A Method Of Determining The Constituents Of Fresh Concrete

BY W. M. DUNAGAN*

DISCUSSION

THE ART of manufacturing concrete has reached a stage at which the quality of a given concrete is definitely established if the actual constituents of the mixture as finally placed are accurately known. The big problem to date has been to control the operations so that the predetermined proportions of the various materials to secure concrete of definite quality, are placed in the mixer. To this end the engineer has had to concern himself with the problem of field control of the entire project, taking the responsibility as to the proportions of materials, the determination of unit weights, making of sieve analyses and moisture determinations, and supervising the details of measurement, mixing, transporting, and placing. The resulting concrete has depended largely upon the skill and determination of the engineer in the execution of each of the above operations. Up to date he has had no way of directly checking the character or uniformity of his product, as placed in the forms, except to await the results of strength tests at some later period.

The test outlined in this paper makes available another solution to this problem of securing a desired quality of concrete. It makes the fullest use of all present day knowledge as to what constitutes concrete of a given quality; it attacks the problem in a direct manner with the maximum economy of time and expense. The solution offered is to take samples of the fresh concrete from the structure at any point and determine the amounts of its significant constituents, namely the water, cement, fine and coarse aggregate. The inspector need not concern himself with volume measurement or moisture determination, nor any of the details

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of *how* the most suitable mixture is obtained, but need only determine that the specified concrete has been placed in the forms and that correct curing procedure follows. The test as outlined here can be run on any batch of concrete in about 15 minutes. The concrete may be tested at any time before it has definitely hardened, a period of several hours after placing. Since on any given job it is not necessary to check up on the material more than once every 2 hours, any competent inspector can handle this test along with his other duties.

The idea of making such a test is far from new; solutions by other methods have been offered by G. S. Griesenauer, R. L. Bertin, and W. I. Freel.* However, the simplicity of the principle and its adaptation to field use in the test shown here makes it of an immediately practical nature.

The principle used throughout is the old Archimedian principle of water displacement, eliminating all necessity for the drying out of any of the constituents. The displacements of the materials are measured from their buoyancy in water. One feature of the test that is of particular advantage is that except for the initial weighing in air which should de done as soon as this sample is selected, the test can be deferred to any time within the setting period, for after the initial weighing of the sample in air any water lost is of no consequence since "water in water weighs nothing."

DETAIL OF PROCEDURE

(1) Take a sample of concrete either from the mixer discharge or from the mass after once in place and determine its weight in air to the nearest gram.

For the apparatus as shown (Fig. 1) a sample of from 3000 to 4000 grams was found to be easily handled; a larger sample may be tested by a series of tests on subdivisions. With scales balanced and the large can "Q" filled to the overflow, place sample in pan on the right beam and weigh by putting weights on left counterbalance beam using slotted weights as indicated.

(2) Determine the weight of this sample immersed in water.

To do this remove container "C," leaving a small amount of water in it and pour the sample into it stirring to remove en-

*M. S. Thesis, Purdue University, 1929.

trained air; allow to settle one minute and immerse in "Q," refilled to overflow level. The difference between this weight and the original weight in air is the buoyancy and is a measure of the absolute volume of the solid material in the sample of concrete.

(3) Remove can "C" and pour sample on nested No. 4 and 100-mesh sieves (Fig. 2) and carefully wash the cement through a 100-mesh sieve.

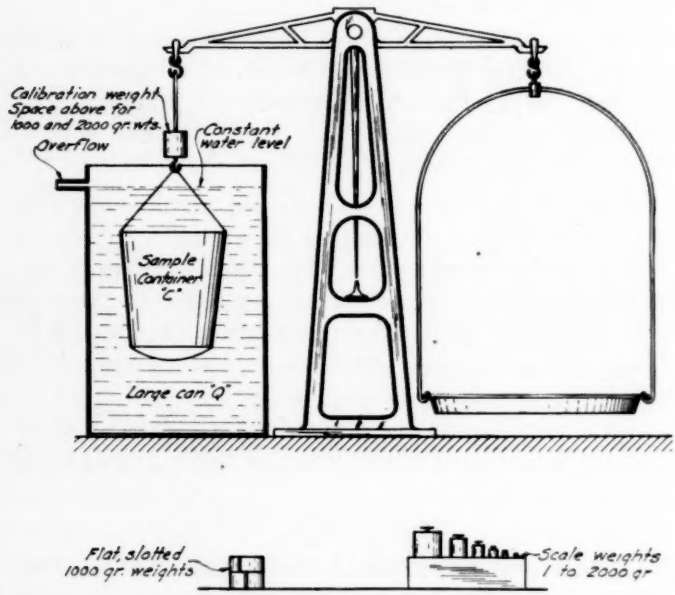


FIG. 1—ARRANGEMENT AND EQUIPMENT OF SPECIFIC GRAVITY APPARATUS, WITH ORDINARY BALANCE SCALE, TO DETERMINE MOISTURE CONTENT OF AGGREGATE AND THE CONSTITUENTS OF A FRESH CONCRETE MIX.

This separates the samples into three parts, cement (through No. 100), fine aggregate between No. 100 and No. 4, and the coarse aggregate retained on No. 4. The small tub shown, is used filled with water but running water will help speed up the separation; no care need be taken to recover the cement or water but care should be taken that *none of the aggregate* is lost.

(4) Determine the immersed weight of the fine and coarse aggregate.

This is done by first weighing the coarse aggregate in container "C" immersed in the can "Q" and then introducing the fine

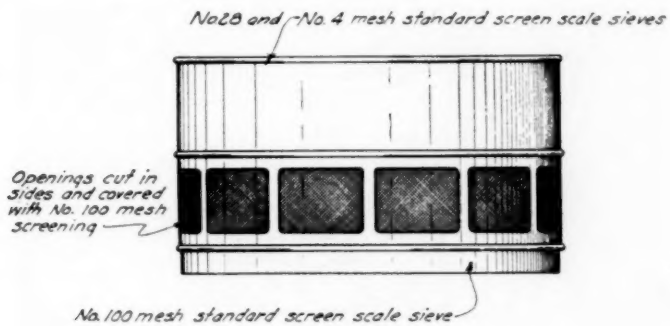

No.8 and No. 4 mesh standard screen scale sieves
Openings cut in sides and covered with No. 100 mesh screening
No. 100 mesh standard screen scale sieve

FIG. 2.

aggregate on top of the coarse (remove "C" to do this) so as to secure the total displacement of both fine and coarse and at the same time the relative amounts of each aggregate. This completes the test. The computations then necessary are slide rule work or simple arithmetic.

The test will be simplified if a scoop-shaped pan is used on the right hand scale pan when the scale is in balance. (Fig. 3) With two of these scoops of exactly the same weight for interchangeability the test can be still further speeded up.

EQUATIONS USED

Note: The difference between the weight immersed and the weight in air is the weight of the displaced water or buoyancy.

Most of the relations in this test will be obvious from the tabulation of Table 1. Only one formula is necessary.

Let X = weight of a material in air

W_s = weight of the material immersed in water

$X - W_s$ = buoyancy

= weight of displaced water

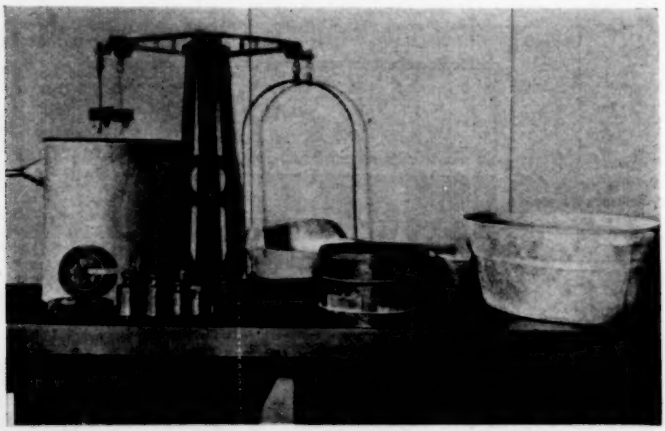
The volume of displaced water = apparent absolute volume of material.

$$\text{Specific gravity} = \frac{\text{weight of the material in air}}{\text{weight of displaced water}} = \frac{X}{X - W_s}$$

from this;

$$X = W_s \left(\frac{\text{Sp. Gr.}}{\text{Sp. Gr.}—1} \right) \quad \ldots\ldots (1)$$

From this formula and the data from the successive weighings the exact quantities of the 3 solid ingredients are determined.



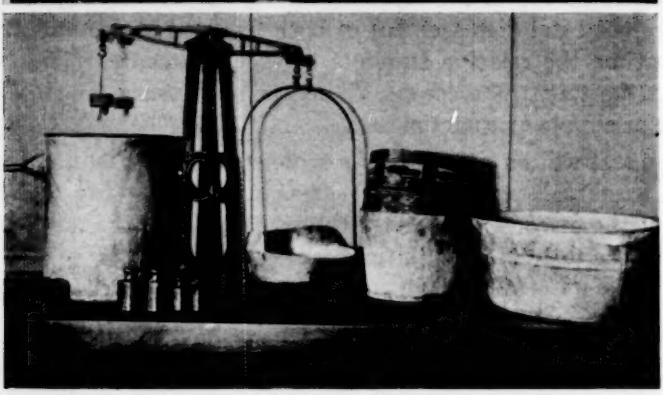


FIG. 3—SCALE BALANCED FOR TEST; NOTE PAN WITH SCOOP ATTACHMENT ON SCALES AND ONE OF DUPLICATE WEIGHT TO INTERCHANGE. SLOTTED WEIGHTS AT LEFT WEIGH 1000 GRAMS EACH. NO. 100 SIEVE HAS MESH IN SIDES AS WELL AS BOTTOM. THIS DEVICE, SIMPLIFIED AND IMPROVED, IS NOW AVAILABLE COMMERCIALLY IN A COMPACT UNIT FOR FIELD USE.

FIG. 4—NOTE CONTAINER "C" REMOVED, OF SUCH SIZE AS TO ALLOW CONTENTS OF INVERTED SIEVES TO BE WASHED INTO IT DIRECTLY. SMALL TUB AT RIGHT FOR WASHING CEMENT THROUGH THE SIEVE.

Its application is brought out in Table I, which gives the results of a particular test.

TABLE I—DETAILED COMPUTATION FOR A TEST

Material Used: Cement Sp. Gr. = 3.10
Sand " " = 2.66
Gravel " " = 2.70

Weight of Sample in Air in Grams			3135
Weight Immersed, Total Sample, Gr.			1848
" "	Sand and Gravel	1555	1555
" "	Sand only	689	
" "	Gravel (diff.)	866	
" "	Cement (diff.)		293

Actual Weight of Materials

Cement (from Eq. 1)	$293 \times \frac{3.10}{3.10-1.00}$	= 433		
Sand	$689 \times \frac{2.66}{2.66-1.00}$	= 1107		
Gravel	$866 \times \frac{2.70}{2.70-1.00}$	= 1377		
Total Solids			2917	2917
Net water used (diff.)				218

$$W/C = \frac{218}{433} = .503 \text{ by weight}$$
$$= .76 \text{ by volume}$$
$$= 5.7 \text{ gals. per sack}$$

Proportion by weight 433:1107:1377
= 1:2.55:3.18

Actual materials used in this batch = 1:2.40:2:97..... $\frac{W}{C}$ = .755 by volume.

The difference between test results and known proportions used indicates the accuracy of the method.

This particular test was run as a demonstration for a group of interested concrete men in the Chicago Laboratory of the Portland Cement Association.

DETERMINATION OF SPECIFIC GRAVITY

In making this test certain definite data must be obtained from the aggregates prior to the test; the care to be taken in obtaining these data depends upon the degree of accuracy with which the

OTHER ANALYSES

This table is typical of other results obtained by this method.

	Using Blended Cement		Using High Early Strength Cement		Using Porous Limestone	
	Mix Used	Analysis	Used	Analysis	Used	Analysis
c	.1	1	1	1	1	1
S	1.22	1.12	2.4	2.3	2.5	2.2
P	1.12	1.85	3.44	3.85	3.2	3.7
W/C	.505	.496	.72	.715	.81	.82

job is to be controlled. The solutions given here were run under conditions allowing very close precision and yet under trying conditions, since in most cases the aggregates were dry and therefore were absorbing water during the progress of test. This condition would not hold in the field as the problem of absorption of at least one material, the sand, is eliminated; in fact, this test is more adaptable to field use with moist aggregates than for laboratory conditions where the material is often dry. The typical problem illustrated in Table I was run on wet material under field conditions. The data necessary for running such a test is as follows:

(1) The specific gravity of the cement. For this test this is best determined in the apparatus shown, rather than by the standard LeChatelier flask. In determining this property of the cement it should be borne in mind that water displacement or buoyancy of the cement varies with the time after which it has been wet. The accompanying curve (Fig. 5) shows how this varies for a typical portland cement. It is necessary to use these data only in case of special investigations as an assumption of 3.10 as the specific gravity of portland cement is usually close enough. This change is due to the absorption of the cement over the period of time, at a given rate. Once this rate has been established the curve may be used to determine the correct value for the time used or some definite time after mixing may be adopted for use.

(2) The specific gravity of the sand and coarse aggregate should be determined in the same manner; the two aggregates

being tested separately since in many cases the sand and stone are of different specific gravities. The specific gravity determined in this manner is the approximate apparent specific gravity as defined by the A. S. T. M.*

In all of the cases shown a typical concrete sand was used. Since for most sands from 2 to 4% by weight pass the 100 mesh sieve and from 1.5 to 2.5% by weight of cement is retained on this sieve, little error will be encountered from this cause except

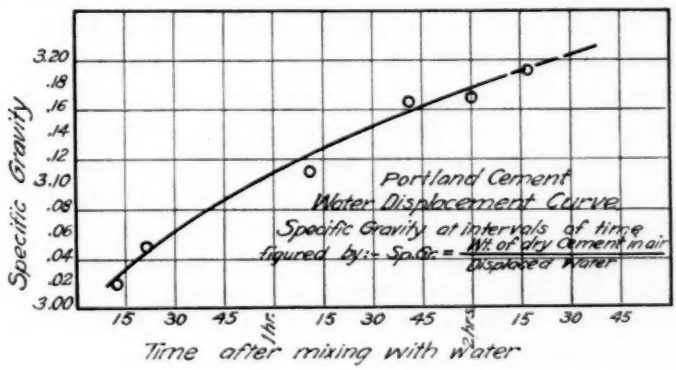


FIG. 5.

in rare cases where correction should be made. Such a correction is easily made if the amounts passing the No. 100 sieve are predetermined, as is usually done in determining the acceptability of the sand for use.

GENERAL NOTES

The entire test may be run under most conditions from the displacement data. In case the aggregates are of uncertain specific gravity or are from a source where this property is known to vary, it is necessary to determine the actual amount of aggregate by drying. However, the displacement of the aggregate *must* be secured since the displacement of the cement is the difference between that of the entire sample and that of the aggregate; since the water in the sample weighs nothing in water, it is automatically eliminated after the first weighing in air.

*Serial Designations C-68, 28T; D30-18

In choosing the sample it was found desirable to weigh a sample as placed on the scales so as to avoid picking over the sample and possibly making it non-representative.

The device as shown was developed in the field on actual construction work; it will be observed that the size of the cans, the weights, sieve openings, and general arrangement have been worked out so as to speed up the test. There are no breakable parts and the whole set-up is permanent and relatively inexpensive. All that is necessary is plenty of water. The data show that this test can be run quickly with results well within the accuracy required of such a test.

It is interesting to note* that this apparatus using the same data as that used for determining the constituents of the fresh concrete provides probably the quickest and most accurate method of making moisture or absorption determinations for field control. The large can "Q" has been designed of such a size that it can be used for making unit weight determinations; the data thus obtained along with the absolute volumes obtained in the second weighing during the analysis of the fresh concrete provides a method of securing void determinations.

With such a test available the question is immediately raised for engineers: From which end of the mixing operation shall we control the quality of the concrete? The segregation test can be run in as short a time as present day moisture determinations and the actual product may be tested in its final position in the structure. One inspector may control several small jobs if it is known that he may take samples from the material at any time, and batches from central mixing plants may be checked by taking samples to nearby laboratories.

This contribution to the problem of field control also brings many research problems, involving the question of uniformity, nearer to solution.

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*See *Engineering News-Record*, p. 738, Nov. 15, 1928.